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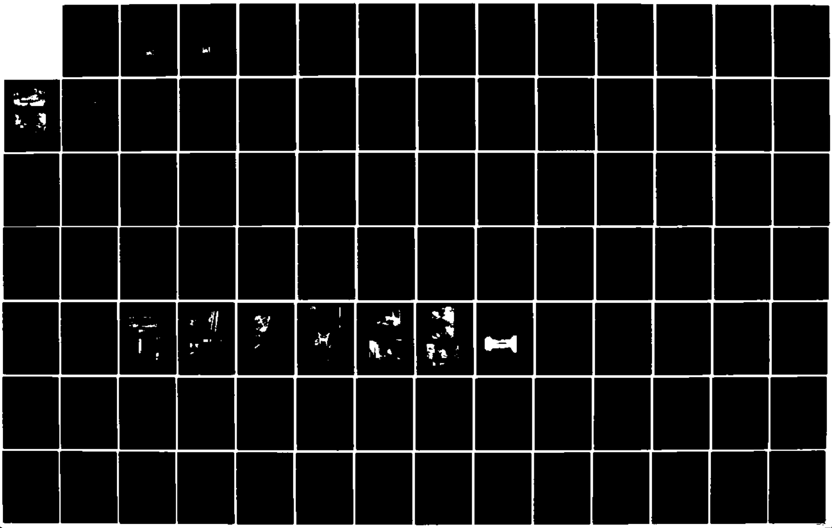

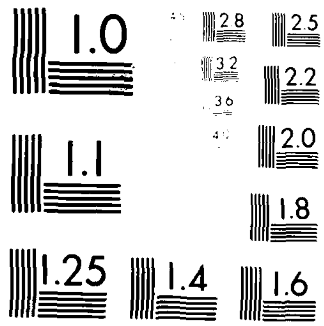

MICROCOPY RESOLUTION TEST CHART

CONNECTICUT RIVER BASIN
ALSTEAD, NEW HAMPSHIRE

VILAS POOL DAM
NH 00009
NHWRB 5.06

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

DECEMBER 1979

(1)

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER NH 00009	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Vilas Pool Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE December 1979
		13. NUMBER OF PAGES 55
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin
Alstead, New Hampshire
Connecticut River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a concrete gravity structure which is 31 ft. high and has an overall span of 78 ft. and is 25 ft. high. The dam is small in size with a high hazard potential. The dam is in poor condition at the present time. Further investigations are recommended to evaluate the hydraulic adequacy of the spillway.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM MASSACHUSETTS 02154

NEDED

MAY 0 5 1980

Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord New Hampshire 03301

Dear Governor Gallen.

Inclosed is a copy of the Vilas Pool Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, Town of Alstead, New Hampshire 02602.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel Corps of Engineers
Division Engineer

Incl
As stated

VILAS POOL DAM
NH [illegible]

Accession For
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CONNECTICUT RIVER BASIN
CHESHIRE COUNTY, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

PHASE I REPORT

Identification No.:	NH 00009
NHWRB No.:	5.06
Name of Dam:	VILAS POOL DAM
Town:	Alstead, New Hampshire
County and State:	Cheshire County, New Hampshire
Stream:	Cold River, A Tributary of the Connecticut River
Date of Inspection:	August 30, 1979

BRIEF ASSESSMENT

The Vilas Pool Dam is located on Cold River, approximately 1 mile upstream of Alstead, New Hampshire. The dam is a concrete gravity structure which is 31 feet high and has an overspan of 78 feet and is 25 feet high. There is a 3 foot square sluice gate at the bottom of this structure.

The dam is owned by the Town of Alstead and is used for recreational purposes. The dam is spanned by a narrow pedestrian bridge which leads to a recreation area on the right bank of the reservoir adjacent to the dam.

The drainage area of the dam covers 62.6 square miles of mountainous woodland with some pasture and minor development. The dam normally impounds 80 acre-feet and has a maximum impoundment of 116 acre-feet. The dam is small in size and its hazard classification is HIGH because of the potential for economic loss and loss of life in up to 10 dwellings in the event of a dam failure.

The test flood for this dam is half the Probable Maximum Flood (PMF). The peak inflow for this flood would be 34,100 cfs which, due to the small amount of storage, results in a maximum outflow of 34,100 cfs. Assuming the gate to be half way open results in a flood stage 16 feet above the spillway crest which overtops the dam by 10 feet. The spillway capacity is not capable of passing the test flood without overtopping the dam. With the reservoir at top of dam elevation (541 feet MSL), the spillway is capable of passing only 12 percent (4,200 cfs) of the routed peak test flood outflow.

The dam is in POOR condition at the present time. Further investigations are recommended to evaluate the hydraulic adequacy of the spillway. Remedial measures to be undertaken by the owner include: patching cracks in the spillway, rehabilitating the left and right abutments, investigating difficult operation of sluice gate, implementing annual maintenance and inspection programs, and developing a downstream emergency warning system.

The remedial measures outlined above should be implemented within one year of receipt of this report by the owner.

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS ANTHONY CAMPAGNA, JR.
No. 21006
CIVIL
STATE OF CALIFORNIA

[illegible]	Nicholas A. Campagna, Jr.
N.H. Registration 322[illegible]	California Registration 21[illegible]

This Phase I Inspection Report on Vilas Pool Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgement and practice, and is hereby submitted for approval.

Joseph W. Finegan

JOSEPH W. FINEGAN, JR., MEMBER
Water Control Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joseph A. McElroy

JOSEPH A. MCELROY, CHAIRMAN
Chief, NED Materials Testing Lab.
Foundations & Materials Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the Test Flood should not be interpreted as necessarily posing a highly inadequate condition. The Test Flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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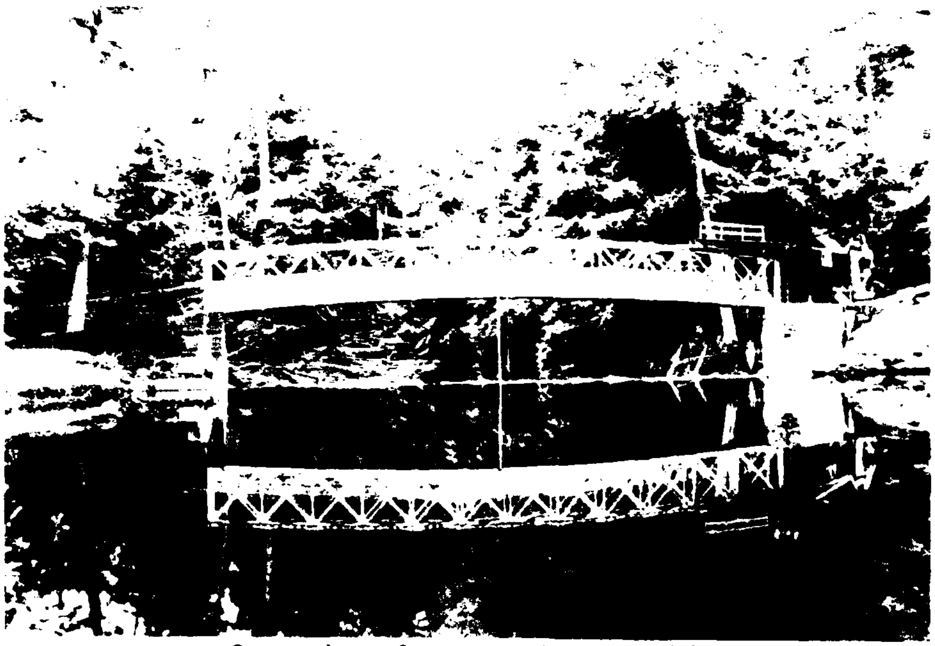
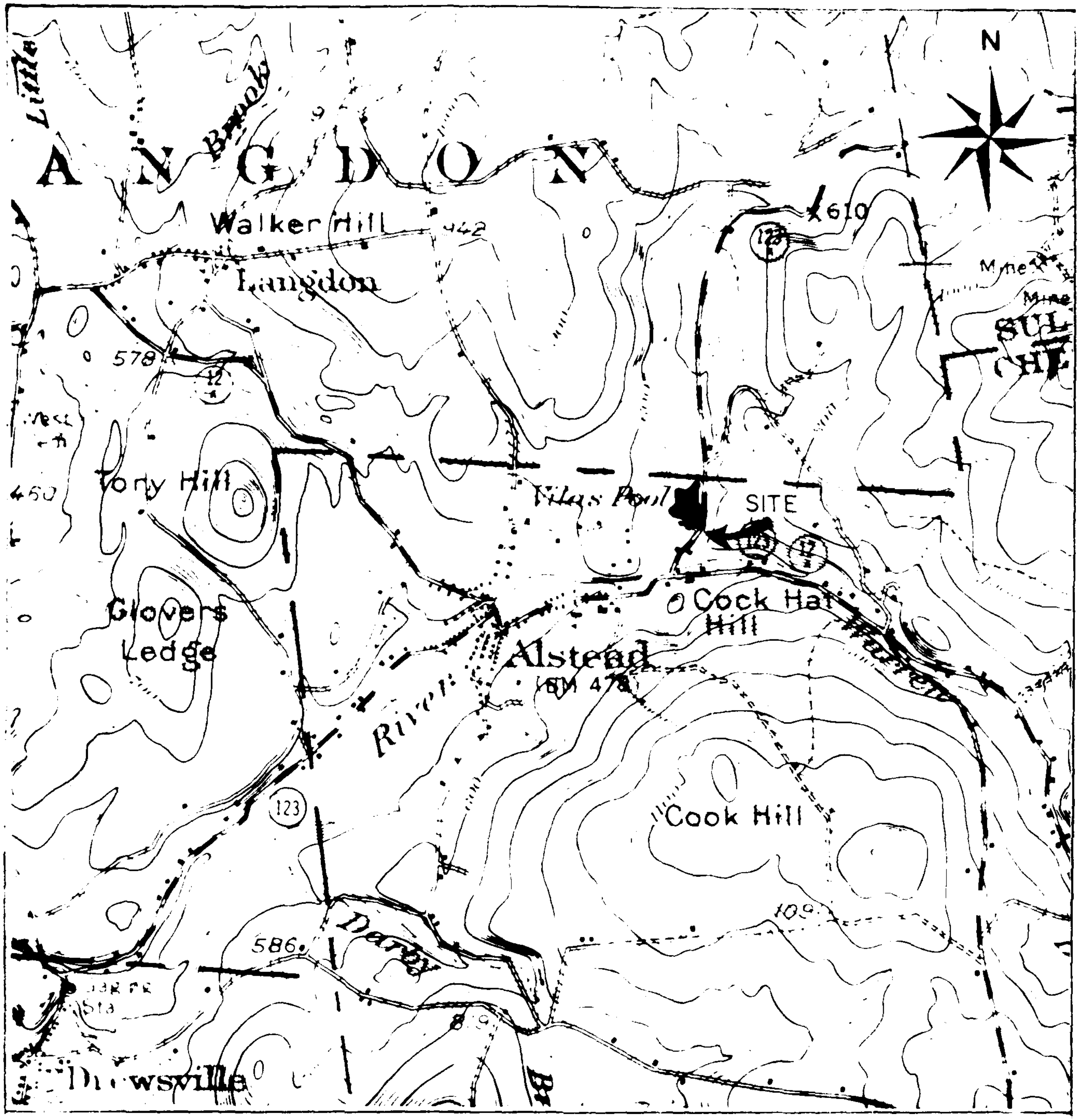


Overview from upstream side

Overview from downstream side



- SCALE -

1/4 1/2 1 MILES

FROM USGS BELLOWS FALLS, N.H. QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC. INC
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS

U.S. ARMY ENGINEER DIV NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS

NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

LOCUS PLAN

VILAS POOL DAM — ALSTEAD, NEW HAMPSHIRE

FILE No. 2327

SCALE AS NOTED

DATE

PHASE I INSPECTION REPORT

VILAS POOL DAM

SECTION 1

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to GZD under a letter of August 28, 1979 from Colonel William E. Hodgson, Jr., Corps of Engineers. Contract No. DACW 33-79-C-0058 has been assigned by the Corps of Engineers for this work.

(b) Purpose

1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.

3) Update, verify, and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams, and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

The Vilas Pool Dam is located on the Cold River approximately 1 mile upstream of Alstead, New Hampshire. It can be reached from state Route 123-A which intersects state Routes 123 and 12 in Alstead. The dam is shown on U.S.G.S. Bellows Falls New Hampshire-Vermont quadrangle at approximate coordinates N43° 9.2' W72° 2[illegible]' (see location map on Page v). Page B-2 of Appendix B is a site plan for this dam.

(b) Description of Dam and Appurtenances

The dam consists of a concrete ogee type gravity spillway, concrete abutments, stone and concrete [illegible] [illegible] concrete wall on the left bank, and a [illegible] [illegible] truss [illegible] bridge spanning the spillway.

There are three controlled outlets in the form of two 24 inch diameter waste gates (which are inoperable) and a 3 foot square sluice gate.

The spillway and both abutments are founded on bedrock. The spillway is 78 feet long and a maximum of 27 feet high.

1 Left Abutment and Concrete Wall

The left abutment consists of a cyclopean concrete gravity structure approximately 9 feet long and 6.5 feet wide with a combination stone and concrete faced extension which ramps to the left bank adjacent to New Hampshire State Highway 123-A. The top of this abutment is 10.5 feet above the spillway crest. The ramp is approximately 3 feet wide and has a pipe rail fence on either side. It is founded on bedrock. The base of this ramp consists of cemented stone masonry which is exposed for 17 feet. The cut off wall on the left bank is [illegible] feet wide.

It originates at the end of the left approach ramp and splays approximately 75 degrees to the left towards the roadway for a distance of approximately 60 feet. The wall then splays towards the impoundment pool at an approximate 30 degree angle for a distance of approximately 20 feet. This concrete wall is constructed with a combination of dry stone masonry, cyclopean concrete and granolithic surface finish. The exposed height of this wall varies from 18 inches adjacent to the ramp to 2 feet at its upstream terminus. This wall is the lowest part of the dam and is considered the top of dam elevation 5[illegible].

(2) Right Abutment

The right abutment is similar in construction as the left abutment consisting of cyclopean structure [illegible].5 feet wide with a ramp extension approximately 27 feet long. The top of this abutment is approximately 10.5 feet above the spillway crest. The abutment and ramp are founded on bedrock. This approach ramp is the access to the recreational facility and is equipped pipe rail fences.

(3) Pedestrian Bridge

The pedestrian footbridge spanning over the spillway consists of 12 equally spaced panels 6 feet, 10 inches long for an overall length of 82 feet. The truss is 8 feet high. The top and bottom chords are X-braced in each panel. The walkway width is approximately 4 feet. A 3-rail pipe fence and chain link fencing 4 feet high is fastened to the truss vertical members on either side of the walkway. Twin inclines angles (back-to-back) are fastened to the top chord of both truss end posts and are anchored in bedrock downstream of the structure for additional lateral stability. A wood framed diving platform has been fastened to the top chord of the truss adjacent to the right bank.

4) Outlets

A sluice gate outlet approximately 3 feet wide and 3 feet high is located at the approximate midpoint of the spillway axis. Records indicate that this opening is controlled by a 3 foot square sluice gate which is mounted on a steel plate frame. To the right of this outlet are two 24 inch diameter sluice gates with inclined hand wheel operators extending to the right downstream bank.

c) Size Classification

The dam's maximum impoundment of 120 acre-feet and height of 31 feet place it in the SMALL size category according to the Corps of Engineers' "Recommended Guidelines".

d) Hazard Potential Classification

The hazard potential classification for this dam is HIGH because of the significant economic losses and the potential for loss of life downstream in up to 10 dwellings in the event of dam failure. Section 5 of this report presents more detailed discussion of the hazard potential.

e) Ownership

The dam is owned by the Town of Alstead, New Hampshire. It is overseen by the Vilas Pool Committee, Town Hall, Alstead, New Hampshire 03602. The chairman of the committee is Merideth Howard who can be reached by telephone at (603) 835-2[illegible].

(f) Operator

The operation of the dam is controlled by the Vilas Pool Committee of Alstead, New Hampshire. Key personnel are as follows:

Merideth Howard, Chairman (603) 835-2[illegible]
Dorothy Blake, Member (603) 835-6[illegible]

Alternatively, the Committee can be reached through the Town Hall on Tuesday or Thursday mornings.

(g) Purpose of the Dam

The purpose of the dam is to impound water for recreational use. The Vilas Pool Recreation Area has been used as a swimming area by local residents.

(h) Design and Construction History

The dam was designed and built by Mr. Ralph D. Carter of Providence, Rhode Island. It was completed in 1925. The left wing wall was rebuilt after the flood of 1927. The two 24 inch diameter gates became inoperable and a 5 foot square gate was added in 1978. This modification was designed by Soils Engineering, Incorporated of Charlestown, New Hampshire and the contractor for this work was Mr. Neil Daniels of Ascutney, Vermont.

(i) Normal Operating Procedure

The dam is normally self regulating. The gate is operated only on an as-needed basis as part of [illegible] maintenance checks.

1.3 Pertinent Data

(a) Drainage Area

The drainage area for this dam covers 62.6 square miles. It is made up primarily of mountainous woodland with some pasture and minor development.

(b) Discharge at Damsite

1) Outlet Works

Normal discharge at the site is through the 5 foot square sluice gate. When the impoundment is high, water flows over the ogee type, concrete spillway which is 78 feet long. The invert of the gate opening is 515 feet (MSL). The elevation of the spillway crest is 535 feet (MSL).

2) Maximum Known Flood

There is no data available for the maximum flood at this damsite.

3) Ungated Spillway Capacity at Top of Dam

The capacity of the spillway with the reservoir at top of dam elevation (541 feet MSL) is 1,240 cfs.

4) Ungated Spillway Capacity at Test Flood

The discharge over the spillway at test flood elevation 551 is 18,470 cfs.

5) Gated Spillway Capacity at Normal Pool

There is no gated spillway.

6) Gated Spillway Capacity at Test Flood

There is no gated spillway.

7) Total Spillway Capacity at Test Flood

The total discharge over the spillway at test flood elevation 551 is 18,470 cfs.

8) Project Discharge at Test Flood

The total project discharge at test flood elevation (551 feet MSL) is 31,100 cfs.

Elevation

1) Streambed at centerline of dam: 510±

2) Maximum tailwater: Unknown

3) Upstream portal invert diversion tunnel: Not Applicable

4) Normal Pool: 535±

5) Full flood control pool: Not Applicable

6) Spillway crest: 535

7) Design surcharge: Unknown

8) Top of dam: 541 (wall on left bank)

9) Test flood design surcharge: 551

(d) Reservoir

1) Length of maximum pool: 1,500± feet

2) Length of recreation pool: 1,000± feet

3) Length of flood control pool: Not Applicable

(e) Storage (acre-feet)

1) Recreation pool: [illegible]

2) Flood control pool: Not applicable

3) Spillway crest pool: 80

4) Top of dam: 116

5) Test flood pool: 175.4

(f) Reservoir Surface (acres)

1) Recreation pool: [illegible]

2) Flood control pool: Not Applicable

3) Spillway crest pool: [illegible]

4) Test flood pool: [illegible]

5) Top of dam: [illegible]

(g) Dam

1) Type: [illegible]

2) Length: 156 feet

3) Height: [illegible] feet

4) Top width: Variable

(h) Diversion and Regulating Tunnel

Not Applicable

(i) Spillways

1) Type: Concrete gravity

2) Length of weir: 78 feet

3) Crest elevation: 555 feet (MSL)

4) Gates: None

5) Upstream channel: Reservoir

(j) Regulating Outlet

The regulating outlet is a 3 foot square sluice gate at elevation 515 feet (MSL).

SECTION 2 - ENGINEERING DATA

2.1 Design Data

The only design data available for this dam is a drawing by Ralph B. Carter, showing plan and cross section of the proposed dam at Vilas Farm. This drawing is contained in the New Hampshire Water Resources Board file on this dam. Design specifications for the sluice gate which was added in 1973 are available in the same file.

2.2 Construction Data

No construction records are available for this dam.

2.3 Operational Record

No operational records are available for this dam.

2.4 Evaluation of Data

(a) Availability

The lack of detailed design and construction data warrants an unsatisfactory assessment for availability.

(b) Adequacy

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of the dam cannot be assessed from the standpoint of reviewing the design and construction data. This assessment is thus based primarily upon the visual inspection, past performance and sound engineering judgement.

(c) Validity

Since the observations of the inspection team generally confirm the information contained in the records of the New Hampshire Water Resources Board, a satisfactory evaluation for validity is indicated.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

(a) General

Vilas Pool Dam is in POOR condition at the present time. Significant repair and maintenance work is necessary to improve the condition of the dam.

(b) Dam

1) Spillway and Outlets (Photos No. 5 and [illegible]

The downstream concrete face of the spillway has been subjected to horizontal cracking and spalling. A horizontal crack is located in line with the r[illegible] of the main sluice gate outlet. This crack is approximately 2 inches wide and 1 inch deep. In addition to these cracks there are numerous horizontal cracks on the downstream face of the structure. Observations of the downstream end of the sluiceway outlet have revealed that this opening was not part of the original construction as evidenced by drill holes in the spillway. This opening was constructed at a later date. Observations also revealed that the outlet has been subjected to a high degree of erosion which has been caused by cavitation and ice damage. In as much as it was difficult to view the tunnel portion of this outlet due to discharging water, the roof of the outlet was distinguished with either an efflorescced surface, exposure of aggregate or a formed surface. The downstream end of the spillway has been subjected to minor surface erosion which can be attributed to ice damage. The outlets for both 24 inch sluice gates consist of cast iron pipe. These pipes are eroded at their inverts which can be attributed to cavitation and ice damage.

2) Left Abutment and Concrete Wall (Photos No. 6,7, and [illegible]

The concrete left abutment including the approach ramp has been subjected to cracking and efflorescence. The [illegible] stream face of the abutment (Photo No. [illegible]) has been [illegible] random horizontal cracks with associated efflorescence and in some cases exudation. [illegible] The downstream face of this structure has also been subjected to horizontal cracks and efflorescence. Two spalls, 8x12 inches and 1x12 inches, both

1 inch deep are located on the downstream corner at the spillway crest. Observations of the interface with the spillway revealed a series of range [illegible] horizontal cracks over the face of this structure. Some of these cracks exhibit a high degree of efflorescence. Observations of the interface between the downstream portion of this structure and [illegible] have revealed that erosion has occurred. This [illegible] can be attributed to ice damage and cavitation [illegible] bedrock at this location is highly fractured. [illegible] No. 6 illustrates a diagonal crack on the [illegible] originating at the reentrant corner of the [illegible] and the approach ramp. This crack follows [illegible] the downstream face. The crack is 1 inch [illegible] the top face and the ramp has settled and [illegible]. Settlement is approximately 1 inch. Another [illegible] crack 1 feet to the left of the first crack [illegible] by settlement. Probings between the [illegible] concrete and the cemented stone masonry have revealed a void 10 feet in length. This void is of variable [illegible] and is up to 2.5 feet deep. The solid concrete facing on the downstream side of this structure precluded further investigation of this void. The railing is in good condition. Observations of the concrete wall on the left bank revealed a series of horizontal joints which relate to capping of this wall. Intermittent transverse cracks on the top surface of the wall can be attributed to temperature stresses. There is no evidence of efflorescence on the wall.

3) Right Abutment (Photos No. 2,3,4 and 5)

The concrete right abutment has been subjected to spalling, cracking and efflorescence on all 3 faces. Erosion and spalling up to 1.5 inches in depth has occurred from approximately 18 inches above the spillway interface to the underside of the steel truss. This erosion can be attriubted to ice damage and spalling due to moisture intrusion subjected to alternate freeze and thaw cycles. Random horizontal cracks with a high degree of efflorescence is also prevalent on this face. The downstream face of this structure has been subjected to spalling in excess of 2 inches over 25% of its surface area which can be attributed to moisture intrusion subjected to alternate freeze and thaw cycles.

The balance of this face of the structure exhibits a high degree of random horizontal cracks with associated efflorescence. Observations of the upstream face of this wall have revealed that erosion has occurred at the normal water line over an area 5 feet long by 1 foot high. This erosion is generally twelve inches deep. The extent of erosion below the water surface could not be determined. This erosion was caused by ice damage. Approximately 10 [illegible] of this face has spalled and in some instances in excess of 5.5 inches deep. In addition to the [illegible] this face is interspersed with random horizontal cracks with efflorescence and exudation. The railing is in good condition.

[illegible] Outlet Structures (Photo No. 9)

The twin 24 inch sluice gate controls are inclined [illegible] degrees from the vertical axis of the spillway and approximately 15 degrees towards the downstream right bank. The service platform for operating the gates is no longer in existence. These gates [illegible] function. The 3 foot square sluice gate is operated by means of an extended aluminum pipe with an attached nut fitting at its base which is connected to a non-rising stem. The top of this pipe, which is approximately 4.5 feet above the footbridge deck, is equipped with a hole for inserting a rod to form a wrench. This pipe wrench can be removed and the gate can be operated from the spillway crest with a conventional tee wrench. A representative of the Town attempted to close the gate with the assistance of inspection personnel. After 30 minutes of futile effort, closing was negligible as evidenced by no apparent reduction in discharge through the tunnel outlet. An attempt was made to open the gate with negative results. Observations concluded that either the gate was binding or underwater obstruction precluded operation. A representative of the Town indicated that a minimal effort was required to operate the gate. The hand wrench, which is stored at the site, when used, did not afford any positive results.

5) Footbridge (Overview Photos)

Observations of the pedestrian footbridge including railings and chain link fencing revealed that the structure was well maintained without any apparent deficiencies.

(c) Reservoir Area (Photos No. 1 and 12)

The shore of the reservoir is generally shallow sloping woodland. It appears stable and in good condition.

(d) Downstream Channel (Photos No. 10 and 11)

The downstream channel is a narrow rock channel through a river valley. It appears stable and in good condition.

3.2 Evaluation

[illegible] good condition of the present [illegible] Problem areas noted during the visual inspection are listed as follows:

(a) Longitudinal cracks in downstream face of spillway.

(b) Cracks and voids in left abutment.

(c) Erosion and spalling of the surface of the right abutment.

(d) Difficult operation of sluice gate.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

No written operational procedures exist for this dam. The dam is normally self regulating. The gate is opened on an as-needed basis for maintenance to the dam or reservoir.

4.2 Maintenance of Dam

No maintenance program exists for the dam. Maintenance is accomplished on an as-needed basis.

4.3 Maintenance of Operating Facilities

No maintenance program exists and maintenance is performed infrequently.

4.4 Description of Warning System in Effect

There is no warning system in effect.

4.5 Evaluation

Additional emphasis on routine maintenance will assist the owners in assuring the long-term safety of the dam and operating facilities. A formal, written, downstream emergency warning system should be developed for this dam.

SECTION 5 - HYDROLOGY/HYDRAULICS

5.1 Evaluation of Features

(a) General

Vilas Pool Dam is a concrete structure on the Cold River in the town of Alstead, 800 feet above the confluence of Warren Brook and the Cold River. The dam is about 4,000 feet upstream of the Route 123 bridge across the Cold River, which is in the center of Alstead. The drainage area upstream of the dam is 62 square miles.

Vilas Pool Dam consists of a 76 foot long, 25 foot high concrete ogee weir placed between two granite ledges. The distance between the ledges narrows to 35 feet at the channel bottom. The dam's concrete abutments are 1[illegible].5 feet above the spillway crest, dropping quickly to [illegible] wall 6 feet above the crest on the left abutment.

(b) Design Data

Data sources available for Vilas Pool Dam include prior inventory and inspection reports. Much of the basic data for the dam is contained in the New Hampshire Water Control Commission's "Data on Dams in New Hampshire" (September 27, 1938) and "Data on Ponds and Reservoirs in New Hampshire" (September 27, 1938), and the New Hampshire Water Resources Board's "Inventory of Dams and Water Power Developments" (1927). Inspection reports dated June 12, 1930 and August 2, 1977 are available, as are early plans of the dam.

Extensive correspondence and plans and specifications for the 1973 addition of a 3 x 3 foot gate are also available, as is a New Hampshire Water Resources Board calculation of the magnitude of the 100-year flood at the dam of 7[illegible] cfs.

(c) Experience Data

No records of flow are known to be available for Vilas Pool Dam. U.S.G.S. gauge 01155000 is located on the Cold River at Drewsville, downstream of the dam. The drainage at this gauge is 83 square miles, compared to 62 square miles at the dam.

The peak flow recorded at the gauge in 39 years of record is 6,710 cfs on December 21, 197[illegible]. Using a drainage area relationship, this would yield a flow of about 5,4[illegible] cfs at Vilas Pool Dam. According to the stage-discharge curve developed in Appendix D, this would have resulted in a peak water surface elevation 0.7 feet above the [illegible] wall on the left bank.

(d) Visual Observations

Vilas Pool Dam is owned by the Town of Alstead and operated for recreation. At the time of the inspection, the pool was closed to the public due to a high bacteria [illegible].

On the left abutment there is a concrete wall with its crest 6 feet above the spillway. Highway 123-A parallels the wall, and provides a low point (elevation 5[illegible] for flow [illegible]5 feet above the dam crest) beyond the end of the wall.

The only operable outlet at Vilas Pool Dam is a 5 feet wide by 3 feet high sluice gate with its invert about 20 feet below the spillway crest operated from the footbridge by a mechanical valve. At the time of the inspection, this gate was slightly open, and was difficult to operate. According to a representative of the town of Alstead, the gate is normally easy to operate.

Two 24 inch conduits through the dam are no longer in operating condition.

For the first 300 feet downstream of the dam, the Cold River runs through a narrow, steep-sided channel. There is no development in this reach. From 300 feet downstream of the dam for the 500 feet to the confluence of Warren Brook and the Cold River, the river passes into a smaller channel with an extensive flood plain. There is a house nine feet above the channel bottom on this reach.

For the 3,200 foot reach from the confluence to the Route 123 bridge, the Cold River runs through a steep-sided channel, with all development 15+ feet above the channel bottom.

Downstream of the Route 123 bridge, which is a concrete structure with an 18 foot by 70 foot concrete opening, the river flattens out considerably, and the channel banks become lower. In the first 1,300 feet below the bridge, there are about 10 houses 8 to 12 feet above the channel bottom.

There is no development for several miles below the reach.

c. Test Flood Analysis

The hydrologic conditions of interest in this Phase I investigation are those required to assess the dam's overtopping potential and its ability to safely allow an appropriately large flood to pass. This requires using the discharge and storage characteristics of the structure to evaluate the impact of an appropriately sized Test Flood. None of the original hydraulic and hydrologic design records are available for use in this study.

Guidelines for establishing a recommended Test Flood based on the size and hazard classification of a dam are specified in the "Recommended Guidelines" of the Corps of Engineers. The impoundment of less than 1,000 acre-feet and the height of less than 40 feet classify this dam as a SMALL structure.

The appropriate hazard classification for this dam is HIGH because of the significant economic losses and potential for loss of life downstream in the event of dam failure. As shown in the Dam Failure Analysis section, the increase in flooding caused by failure would pose a threat to property and to lives along the Cold River in Alstead (see Dam Failure Analysis section).

As shown in Table [illegible] of the Corps of Engineers' "Recommended Guidelines", the appropriate Test Flood for a dam classified as SMALL in size with a HIGH hazard potential would be between one-half the Probable Maximum Flood (PMF) and the PMF. For situations in which a range of possible Test Flood flows is given, the "Recommended Guidelines" indicate that the value most closely related to the dam's hazard classification should be used. Since Vilas Pool Dam is on the low side of HIGH hazard, the [illegible] test flood is one-half the PMF. Using the Corps of Engineers, New England Division "Maximum Probable Flood Peak Flow Rates" for a drainage area of 62 square miles with rolling topography yields a peak one-half PMF inflow of 550 csm, or 34,100 cfs. Assuming that the gate is one-half open, this results in a stage 16 feet above the spillway crest, 10 feet above the wall to the left, and 7.5 feet above the top of the abutments. This flow is 7.8 times greater than the spillway capacity of 4,4[illegible] cfs with water surface at top of dam.

Dam Failure Analysis

The peak outflow that would result from the failure of Vilas Pool Dam is estimated using the procedure suggested in the Corps of Engineers, New England Division's April 1978 "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs". Failure is assumed to occur with the water surface elevation at the top of the wall on the left abutment at 541 feet MSL, 6.0 feet above the spillway crest. The top of this wall corresponds most closely to the "top of dam" failure assumption generally used. (The spillway crest elevation is estimated as 535 feet MSL from a U.S.G.S. topographic map.)

The discharge just prior to failure at this elevation is given by the Stage-Discharge curve developed in Appendix D as 4,100 cfs. The tailwater prior to failure at this discharge is estimated to be 10 feet of flow.

The most likely failure mode for Vilas Pool Dam is the destruction of the concrete spillway. For these calculations, that spillway is assumed to be removed upon failure. The resulting increase in flow would be 7,500 cfs or a total of about 11,900 cfs. This would increase the tailwater from 10 feet to about 18 feet [illegible].

The first downstream development affected by dam failure would be a house 9 feet above the streambed about 500 feet downstream of the dam. The dam failure flow of 11,900 cfs would increase the stage from 7 to 11 feet at this location, causing flooding and threatening loss of life. This level of flow would be two feet above the first floor level of the house.

The only other development threatened by dam failure flows is from 1,0[illegible] to 5,[illegible] feet downstream of the dam, [illegible] below the Route 12[illegible] bridge. Inflow from Warren Brook would increase the pre-failure flow to an estimated 7,[illegible] cfs in this reach. The reach contains about 10 houses [illegible] to 12 feet above the channel bottom.

Peak dam failure flows would range from 9,000 cfs at the upstream end of the reach to 8,500 cfs downstream. This would cause the stage to increase from 10.5 feet to about 12 feet, increasing flood damages at the houses significantly. However, the threat of loss of life would not be great, due to the flooding prior to dam failure and the relatively small increment in flood depth.

Downstream of this reach, the Cold River runs through several miles with no development on the stream. Dam failure flows would be attenuated in this reach. The chart on the next page summarizes the downstream effects of the failure of Vilas Pool Dam.

IMPACT OF DAM FAILURE [illegible]

Location	Distance Downstream of Dam (ft)	Number of Dwellings	Level Above Streambed (ft)	Flow and Stage Before Failure	Flow and Stage After Failure	Comments
Tailwater	-			1,100 cfs 10 feet	11,900 cfs 18 feet	
house, about 500 feet downstream of dam	500	1	0	1,100 cfs 7 feet	11,900 cfs 11 feet	danger of loss of life, [illegible]
just upstream of Warren Brook	300			1,100 cfs 7 feet	11,300 [illegible] 11 feet	
just downstream of Warren Brook	300		-	5,100 cfs 8 feet	12,000 cfs 13 feet	
just upstream of Route 123 bridge	1000			5,100cfs 8 feet	9,000 cfs 12.5 feet	
just downstream of Route 123 bridge	1000			5,100 cfs 10-11 feet	9,000 cfs 12 feet	
1,200' downstream of Route 123 bridge	5300	100	8-12	5,100 cfs 10-11 feet	8,500 cfs 12 feet	Slight damage of [illegible]

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observation

1) Spillway

This concrete structure is in fair condition with the exception of the longitudinal cracks located on either side of the roof of the sluice gate outlet tunnel. This outlet has been subjected to a high degree of erosion.

2) Left Abutment

This concrete structure has been subjected to spalls, random horizontal cracks with a high degree of efflorescence and some exudation. Erosion has occurred between the downstream interface of the abutment and the rock outcrop. Bedrock at this location is highly fractured. The approach ramp has settled and rotated away from the abutment resulting in a pronounced diagonal crack entirely through the structure. An additional sloping crack is located on the upstream face of the platform wall. A void approximately 10 feet long and 2.5 feet deep is located at the interface of the concrete platform and the stone masonry foundation. This void is of variable width.

3) Right Abutment

This concrete structure has been subjected to a high degree of horizontal cracks, efflorescence, exudation and spalls. The spalling over the downstream face is in excess of 25% of its surface area. The balance of this face exhibits a high degree of random horizontal cracks and efflorescence. Erosion and spalling is prevalent over most of the interface with the spillway. Erosion has occurred at the water line of the upstream face in the magnitude of 5 feet long, at least 12 inches high and 12 inches deep. Approximately 10% of this face has spalled and in some instances up to 5 inches deep. This face is deep. This face is also interspersed with random horizontal cracks, efflorescence and exudation.

4) <u>Sluice Gates</u>

The twin 24 inch sluice gates are no longer functional. The 3 foot square sluice gate is extremely difficult to operate due to binding or obstructions.

5) <u>Pedestrian Footbridge</u>

This structure is in good condition.

<u>Design and Construction Data</u>

No plans or calculations of value to a stability assessment available for this dam.

<u>Operating Records</u>

There are no known operating records for this dam.

<u>Post Construction Changes</u>

In 1973 the 3 foot square sluice gate was added. This did not adversely effect the stability of the dam.

<u>Seismic Stability</u>

The dam is located in Seismic Zone No. 2 and, in accordance with Phase 1 guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL ASSESSMENT

7.1 Dam Assessment

a. Condition

The dam is in POOR condition at the present time.

b. Adequacy of Information

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of the dam cannot be assessed from the standpoint of reviewing design and construction data. This assessment is thus based primarily on the visual inspection, past performance, and sound engineering judgement.

c. Urgency

The engineering studies and improvements described herein should be implemented by the owner within one year of receipt of this Phase I Inspection Report.

d. Need for Additional Investigations

Additional investigations should be carried out as outlined in Paragraph 7.2 below.

7.2 Recommendations

It is recommended that the town of Alstead retain the services of a registered professional engineer to perform more detailed hydrologic and hydraulic studies to determine the need for additional spillway capacity.

7.3 Remedial Measures

It is recommended that the owner institute the following remedial measures:

1. Patch longitudinal cracks in the concrete on the downstream face of the spillway.

2) Pressure grout cracks and voids in concrete in left abutment. Clean downstream base and pack with high strength mortar.

3) Clean and repair concrete in right abutment.

4) Investigate the cause of the difficult operation of the main sluice gate.

5) Implement and intensify a program of diligent and periodic maintenance.

6) Implement a program of annual technical inspections including operation of all outlet works.

7) Develop a formal written downstream emergency warning system.

7.4 Alternatives

Breaching the dam is a possible alternative to the above measures.

APPENDIX A

INSPECTION CHECKLIST

INSPECTION TEAM ORGANIZATION

Date: August 30, 1979

Project: NH 00039
VILAS POOL DAM
Alstead, New Hampshire
NHWRB 5.06

Weather: Cloudy, [illegible]

INSPECTION TEAM

Nicholas A. Campagna	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD)	Team Captain
M. [illegible]	GZD	[illegible]
Jeffrey M. Hardin	GZD	Soils
Andrew Christo	Andrew Christo Engineers (ACE)	Structural
Paul Ranch	ACE	Structural
Carl [illegible]	ACE	Structural
Richard Laramie	Resource Analysis, Inc. (RAI)	Hydraulics
Tom G[illegible]	RAI	Hydraulics

Owner's Representative Present

Meredith Howard, Chairman, Vilas Pool Committee
Dorothy Blake, Vilas Pool Committee
Gary Kerr, New Hampshire Water Resources Board
NHWRB Representative Present

CHECK LISTS FOR VISUAL INSPECTION

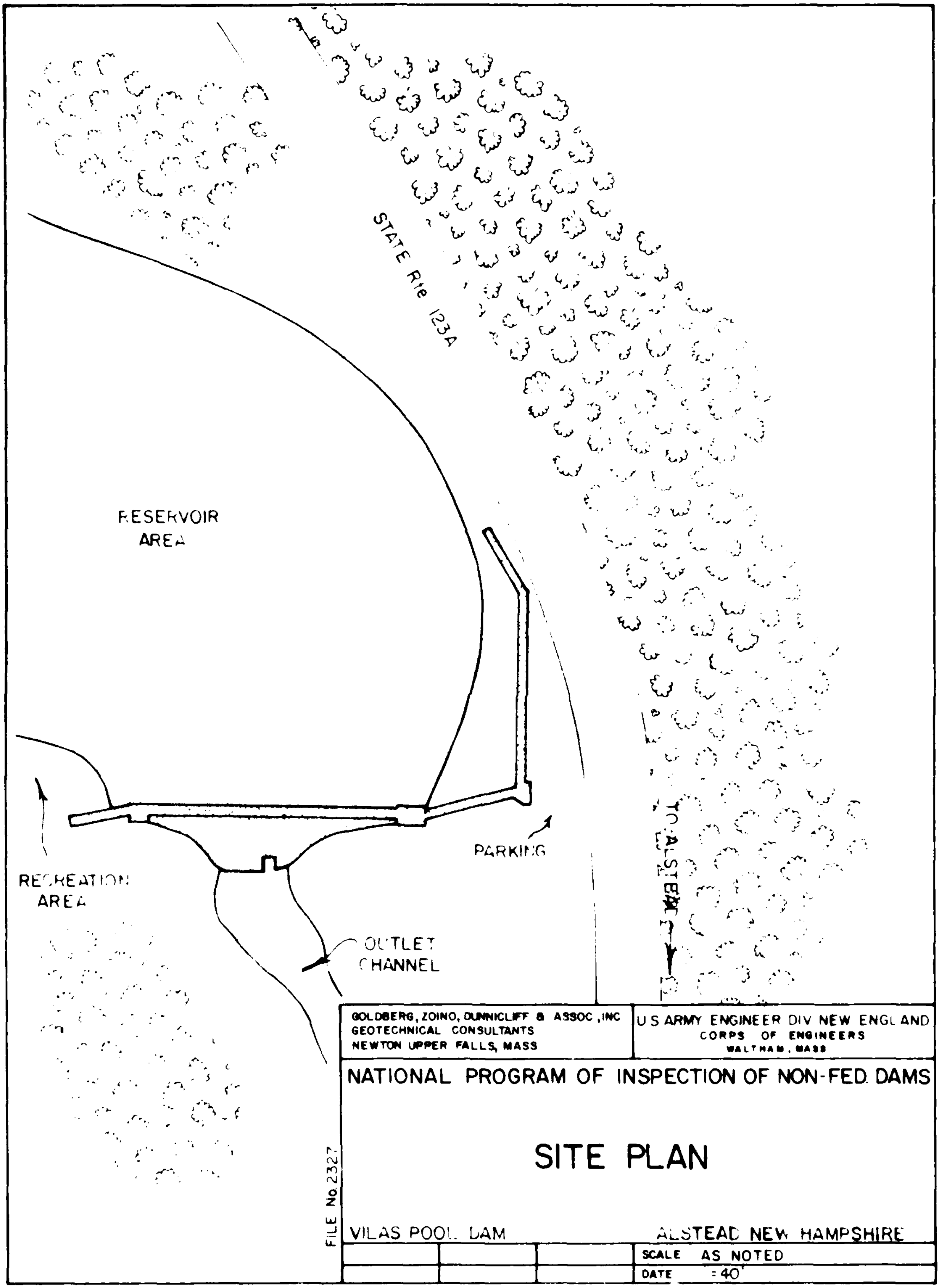
AREA EVALUATED	BY	CONDITION & REMARKS
GENERAL		
Crest Elevation	JMH	5[illegible] feet
Current Pool Elevation	JMH	[illegible] feet
SPILLWAY		
[illegible]	AC	Fair
		See [illegible]
		Downstream [illegible]
[illegible]		Horizontal crack [illegible] right and [illegible] left [illegible] x 2" wide x 1' deep both sides of [illegible] sluice outlet. Random horizontal cracks elsewhere.
[illegible]		None Noted
[illegible]		None Noted
[illegible]		None Noted
III. ABUTMENTS		
General Condition		Fair
Spalling		Downstream corner above spillway crest 8' x 12" and 4" x 11 [illegible] 2" deep.
Erosion		At downstream interface with bedrock.
Cracking	AC	Numerous horizontal cracks on D face. Diagonal crack through abutment (1") and diagonal crack on approach ramp.

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Rusting or Staining of Concrete	AC	Abutment face under bottom chords of truss.
Visible Reinforcing		None Noted
Efflorescence		Upstream face and interface with spillway.
Erosion		Interface with Spillway.
Undermining		Void under approach ramp 2 feet long up to 2.5 feet in depth.
Rating		No deficiencies noted.
LEFT CONCRETE WALL		
Condition of Concrete		Fair
Spalling		None Noted
Erosion		None Noted
Cracking		Minor horizontal construction joints and minor intermittent transverse cracks.
Rusting or Staining of Concrete		None Noted
Visible Reinforcing		None Noted
Efflorescence		None Noted
RIGHT ABUTMENT		
Condition of Concrete		Poor
Spalling	AC	25% of downstream face in excess of 2". Interface with pier up to 1.5" deep. 10% of upstream face spalled up to 5.5" deep.

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Erosion		Erosion at interface with Spillway. Upstream face 5' long by 12" high by 12" deep.
Cracking		Random horizontal cracks on 3 faces.
Rusting or Staining of Concrete		Abutment face under bottom chords of truss.
Visible Reinforcement		None Noted
Efflorescence		All 3 faces - severe.
Exudation		All 3 faces.
Ladder		No deficiencies noted.
SLUICE GATE		
12" Square Gate		Negative results during operation - binding or [illegible] water obstruction.
Twin 24-inch Gates		No longer operable.
Footbridge and Railings		No deficiencies noted.

APPENDIX B



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GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS

U S ARMY ENGINEER DIV NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS

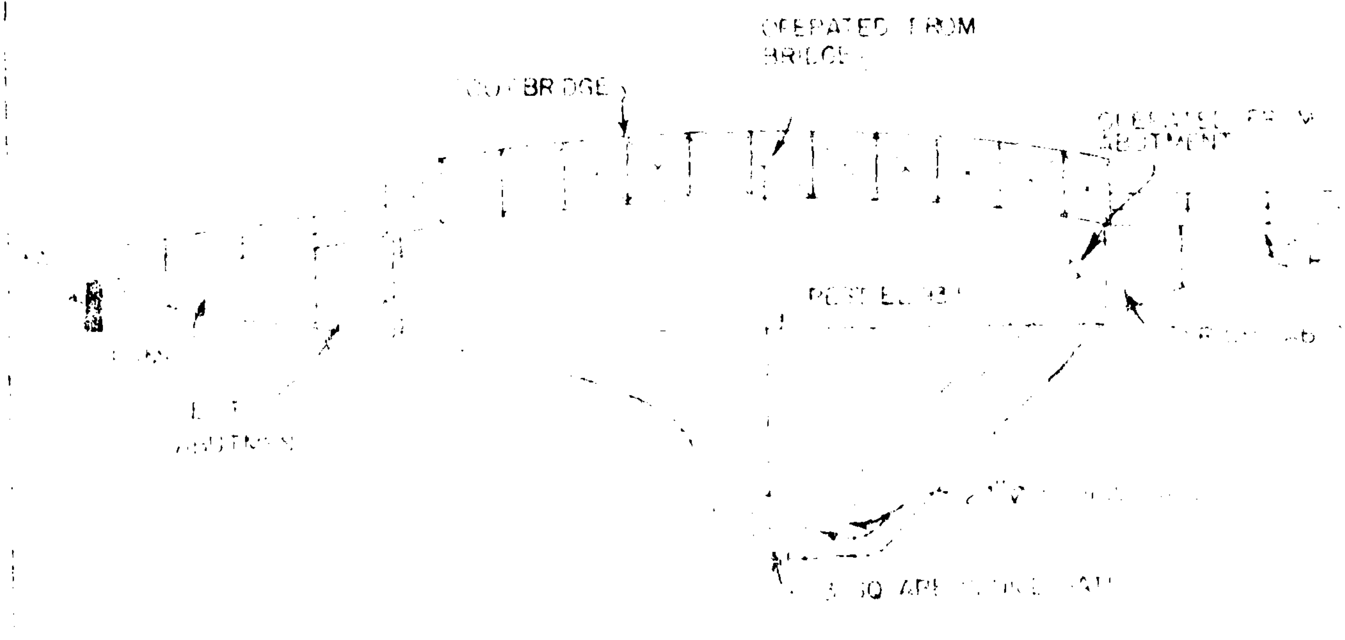
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

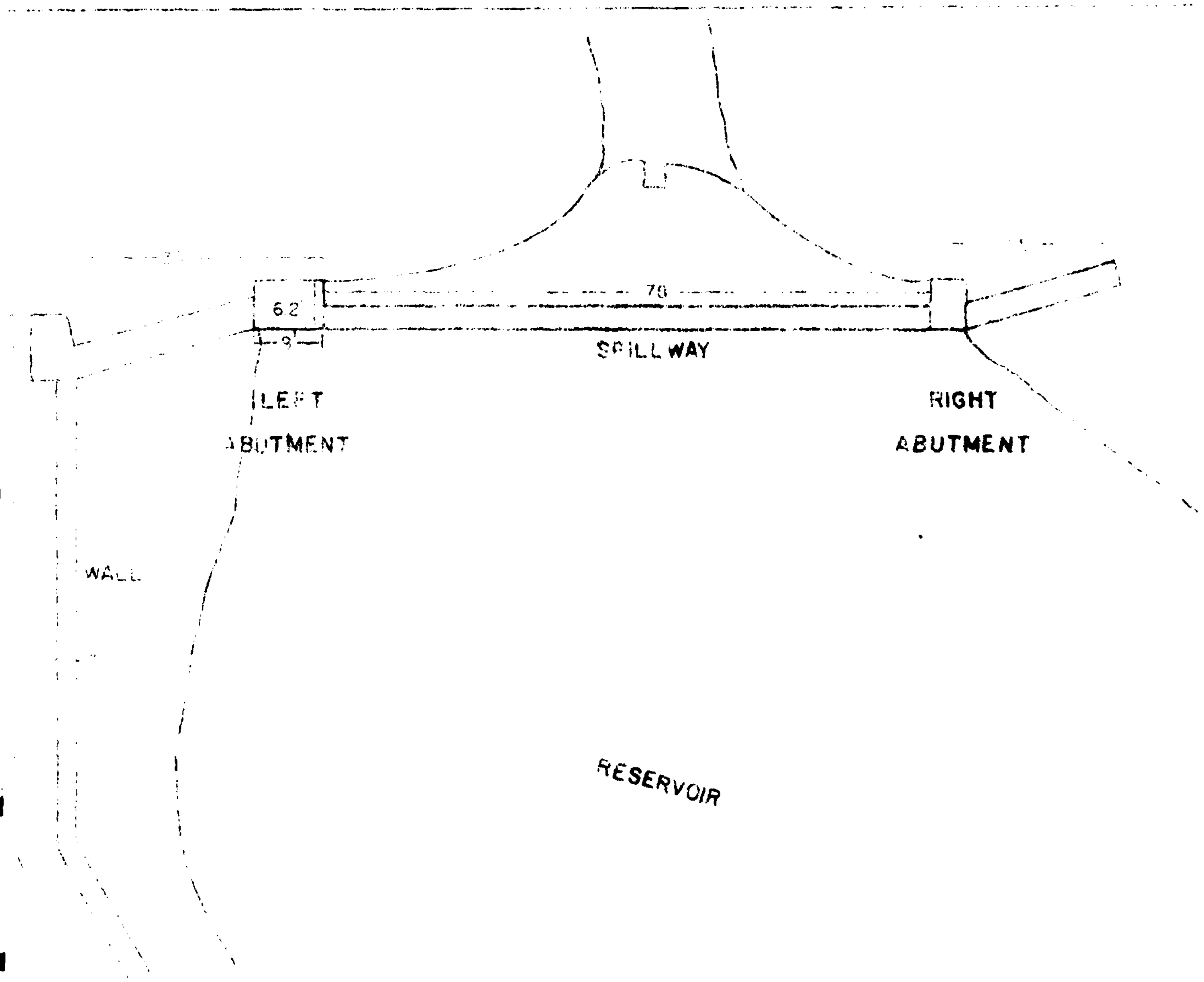
SITE PLAN

VILAS POOL DAM — ALSTEAD NEW HAMPSHIRE

SCALE AS NOTED

DATE =40'


FROM
BRIDGE
ABUTMENT



PLAN VIEW

GEOTECHNICAL ...
NEWTON ...

CORPS OF ENGINEERS

NATIONAL PROGRAM OF INSPECTION OF NON-FED...

ELEVATION AND PLAN VIEWS

... DAM

... STEAD, NEW HAMPSHIRE

SCALE AS NOTED

DATE

NEW HAMPSHIRE WATER RESOURCES BOARD

INVENTORY OF DAMS AND WATER POWER DEVELOPMENTS

DAM

BASIN Connecticut NO. 7 [illegible] ~~[illegible]~~
RIVER [illegible] MILES FROM MOUTH [illegible] 71
TOWN Alstead OWNER ~~[illegible]~~, [illegible]
LOCAL NAME OF DAM [illegible]
BUILT 1925 DESCRIPTION Concrete, [illegible]

[illegible] AREA-ACRES [illegible] FT. [illegible] CAPACITY-ACRE FT.
HEIGHT-TOP TO BED OF STREAM-FT. 17.5 35 MAX. MIN.
OVERALL LENGTH OF DAM-FT. ~~[illegible]~~ 136 [illegible] HEIGHT ABOVE CREST-FT.
PERMANENT CREST ELEV. [illegible] LOCAL [illegible]
TAILWATER ELEV. [illegible] LOCAL [illegible]
[illegible] [illegible] [illegible]-FT. [illegible]
[illegible]
[illegible]
[illegible] [illegible] [illegible]

REMARKS Condition [illegible]
[illegible]

[illegible]

[illegible]

[illegible] Swimming [illegible]

REMARKS [illegible] Feb. [illegible]
P.F. [illegible]
[illegible]

[illegible]

NEW HAMPSHIRE WATER CONTROL COMMISSION
DATA ON DAMS IN NEW HAMPSHIRE

LOCATION STATE NO. [illegible]

Town [illegible] : County Cheshire

Stream Cold River(Vilas Pool)

Basin-Primary Conn : Secondary Cold River

Local Name Vilas Pool

Coordinates—Lat. [illegible] : Long. [illegible]

GENERAL DATA

Drainage area: Controlled .... Sq. Mi.: Uncontrolled .... Sq. Mi.: Total [illegible] Sq. Mi.

Overall length of dam [illegible] ft.: Date of Construction [illegible]

Height: Stream bed to highest elev. [illegible] ft.: Max. Structure [illegible]

Cost—Dam .... : Reservoir ....

DESCRIPTION C See Face—— Concrete

Waste Gates

Type ....

Number .... : Size .... ft. high x .... ft. w.

Elevation Invert .... : Total Area .... sq. ft.

Hoist ....

Waste Gates Conduit

Number [illegible] : Materials ....

Size .... ft.: Length .... ft.: Area [illegible] sq. ft.

Embankment

Type [illegible]

Height—Max. .... ft.: Min. ....

Top—Width .... : Elev. ....

Slopes—Upstream .... on .... : Downstream .... on ....

Length—Right of Spillway .... : Left of Spillway ....

Spillway

Materials of Construction Stone & Concrete

Length—Total .... ft.: Net [illegible]

Height of permanent section—max. .... ft.: Min. ....

Flashboards—Type .... : Height ....

Elevation—Permanent Crest .... : Top of Flashboard ....

Flood Capacity [illegible] cfs.: .... cfs sq. mi.

Abutments

Materials: [illegible]

Freeboard: Max. [illegible] ft.: Min. .... ft.

Headworks to Power Devel.—(See "Data on Power Development")

OWNER [illegible]

REMARKS [illegible]

Tabulation By [illegible] Date [illegible]

NEW HAMPSHIRE WATER CONTROL COMMISSION

DATA ON RESERVOIRS & PONDS IN NEW HAMPSHIRE

LOCATION — AT DAM NO. 5.08

Town: Alstead; County: Cheshire

Stream: Cold River

Basin—Primary: Conn.; Secondary: Cold River

Local Name: Vilas Pool

DRAINAGE AREA

Controlled ...... Sq. Mi.: Uncontrolled ...... Sq. Mi.: Total 71 Sq. Mi.

ELEVATION vs. WATER SURFACE AREA vs. VOLUME

	Head Feet	Surface Area Acres	Volume Acre Ft.
(1) Max. Flood Height			
(2) Top of Flashboards			
(3) Permanent Crest			
(4) Normal Drawdown			
(5) Max. Drawdown			
(6) Original Pond			

Base Used ......: Coeff. to change to U.S.G.S. Base ......

RESERVOIR CAPACITY

	Total Volume	Useable Volume
Drawdown	ft.	ft.
Volume	ac. ft.	95 ac. ft.
Acre ft. per sq. mi.		
Inches per sq. mi.		

USE OF WATER: Swimming and Boating

OWNER: Town of Alstead

REMARKS

Tabulation By A. [illegible] Date Sept. [illegible], 19[illegible]

5.06

Alstead (Cheshire County)   Inspected June 12, 1930.

Vilas Pool Dam - Charles N. Vilas

This is a concrete dam built in 1925, ogee type, built between ledges, usually overflowing. Gates work O. K mechanically. Chains and life buoy are placed across dam in order to prevent any loss of life as the lake is used for boating. Two weeks before inspection the gates had been opened to put up the chains so that no water overflowed and at that time there was no seepage in the dam according to Mr. Prentiss, the caretaker.

In the flood of 1927 a section of the wing wall near the highway was taken out. This was refilled by large stones and a new retaining wall, reenforced concrete laid November, 1927. The wall is three feet wide and averages six feet deep. There is a small seepage on the downstream side of the fill where the flood waters rushed through, but part of this is no doubt due to a spring on the hillside above where a road drain comes across. The dam is in good condition. The foot bridge abutments are slightly cracked.

DIVI-4
DIVI-5.

NEW HAMPSHIRE WATER RESOURCES BOARD

INSPECTION REPORT

Town: Alstead Dam Number: 5.06

Name of Dam, Stream and/or Water Body: Vilas Pond

Owner: Town Telephone Number:

Mailing Address:

Max. Height of Dam: 13' Pond Area: 7 A Length of Dam: 100'

FOUNDATION: Ledge

OUTLET WORKS: 73' o-g Type concrete spillway

6' Freeboard

2 - 24" dia waste gate

1 36" x 36" pond drain

ABUTMENTS: Concrete in good shape

EMBANKMENT:

Note: Give sizing, condition and detailed description for each item, if applicable.

SPILLWAY: Length: 72 Freeboard: 10' will go over Road @ 6

SEEPAGE: Location, estimated quantity, etc.

None

Changes Since Construction or Last Inspection:

[illegible] Condition:

Overall Condition of Dam: Good

Contact With Owner: NC

Date of Inspection: 2 Aug 82 Suggested Reinspection Date:

Class of Dam: [illegible]

Signature [illegible]

Date

[illegible] Condition and detailed description [illegible]

[illegible]

[illegible]

[illegible]

[illegible] Alstead Water Board [illegible]

SITE LOCATION [illegible] Alstead

[illegible] Vilas Pond [illegible] Cold River

[illegible]

[illegible] (PROPOSED, EXISTING) [illegible] 18' [illegible] 100'

[illegible] (PROPOSED, EXISTING) [illegible] Concrete O-G spillway

[illegible] 76

[illegible]

[illegible]

Town of Alstead

[illegible]

[illegible] DAMAGE [illegible] IN DETAIL AND INCLUDE ANY [illegible]

[illegible] damage to buildings in Town

[illegible]

CLASS OF STRUCTURE [illegible] A [illegible] 5.00

DATE OF INSPECTION 2 Aug 77

SIGNED S Buckley

[illegible]

PERTINENT DATA NOT INCLUDED

The New Hampshire Water Resources Board (NHWRB) maintains a file on this dam. Included in this file are:

(1) Correspondence from [illegible] to 1975 pertaining to the addition of the 3 foot square sluice gate.

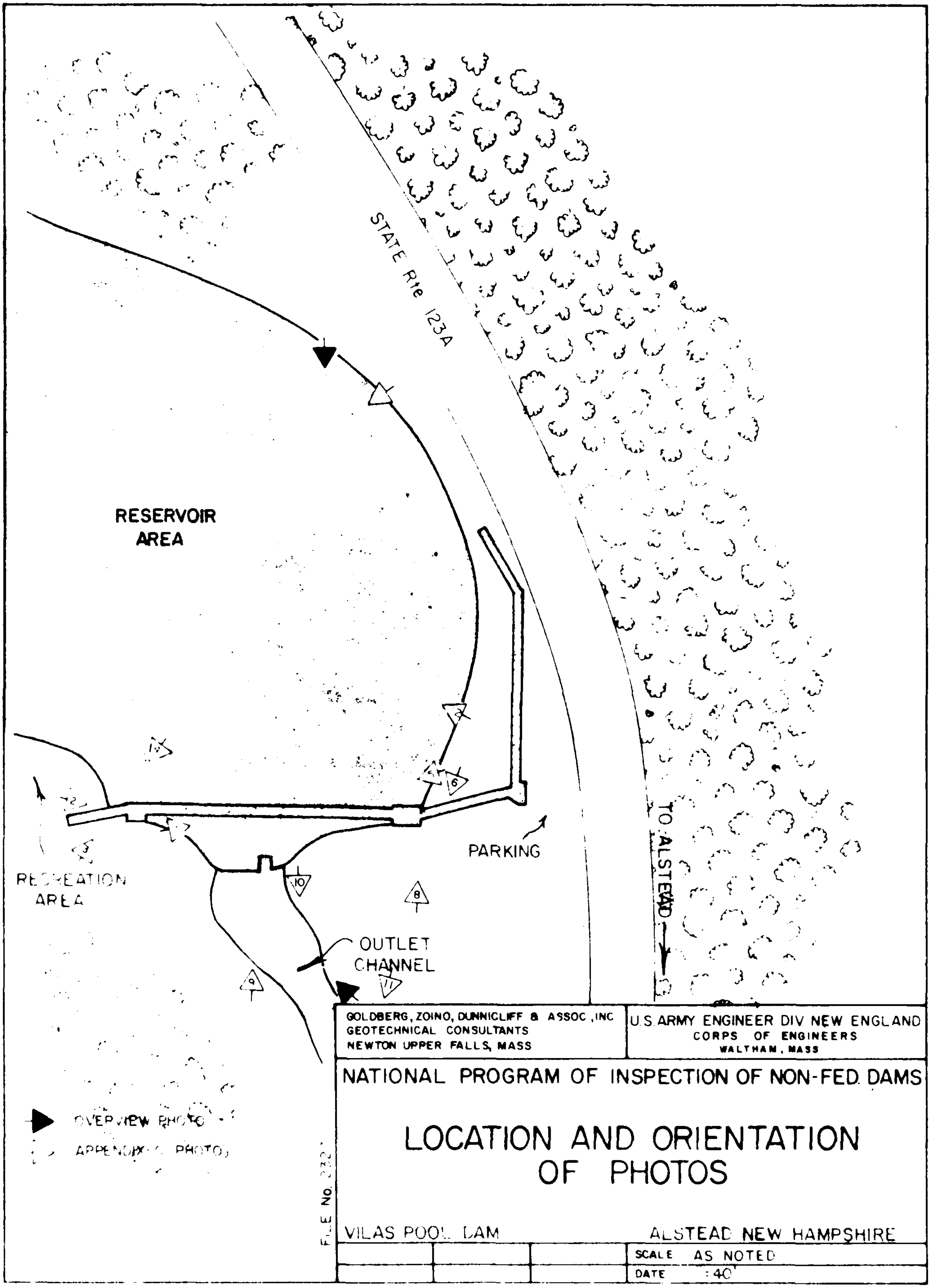
(2) Specifications for sluice gate installation by Soils Engineering, Incorporated.

(3) Design drawings of gate installation.

(4) Water Control Commission Form concerning [illegible] the [illegible] file.

APPENDIX C

DEFINITIONS



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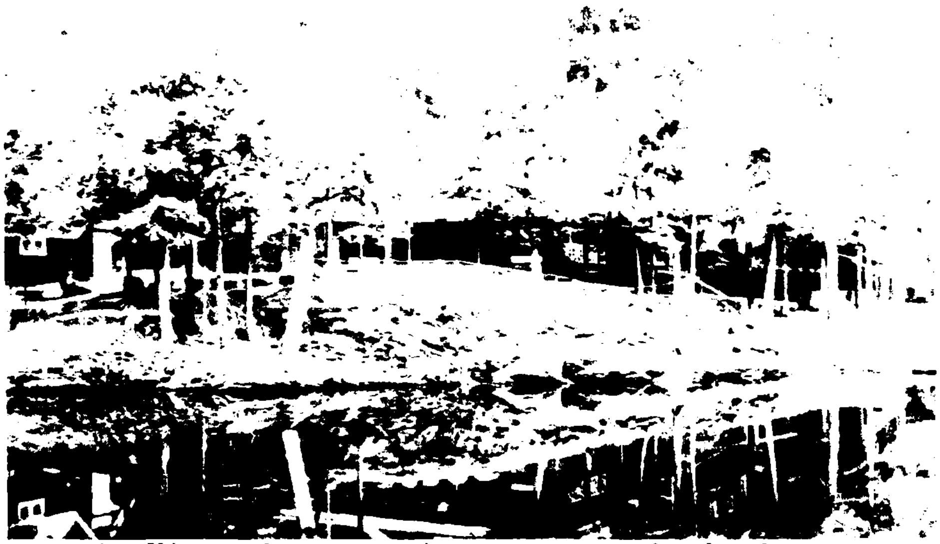
WALTHAM, MASS

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION AND ORIENTATION OF PHOTOS

VILAS POOL DAM — ALSTEAD NEW HAMPSHIRE

SCALE AS NOTED
DATE :40'



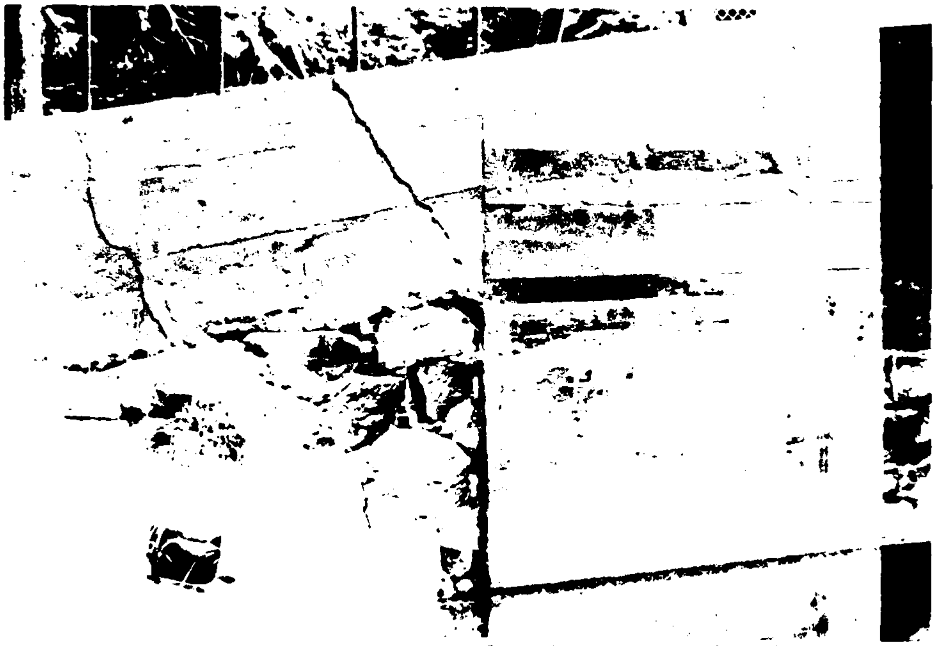
1. View of recreation area on the bank to the right of the dam

2. View of the right wall of the right abutment showing spalling, efflorescence and cracking

3. Downstream wall of right abutment showing spalling and efflorescence

4. Upstream side of right abutment showing spalling and efflorescence

5. Crest and right abutment showing hand wheel controls for the two 24 inch gates



6. Upstream face of left abutment showing diagonal cracking, efflorescence, and exudation

7. Right side of left abutment showing cracking, efflorescence, and exudation

8. Downstream side of left abutment showing cracking and bedrock

9. Downstream face of spillway showing outlets of two 21 inch diameter gates and the sluice gate

10. Downstream channel immediately below the dam

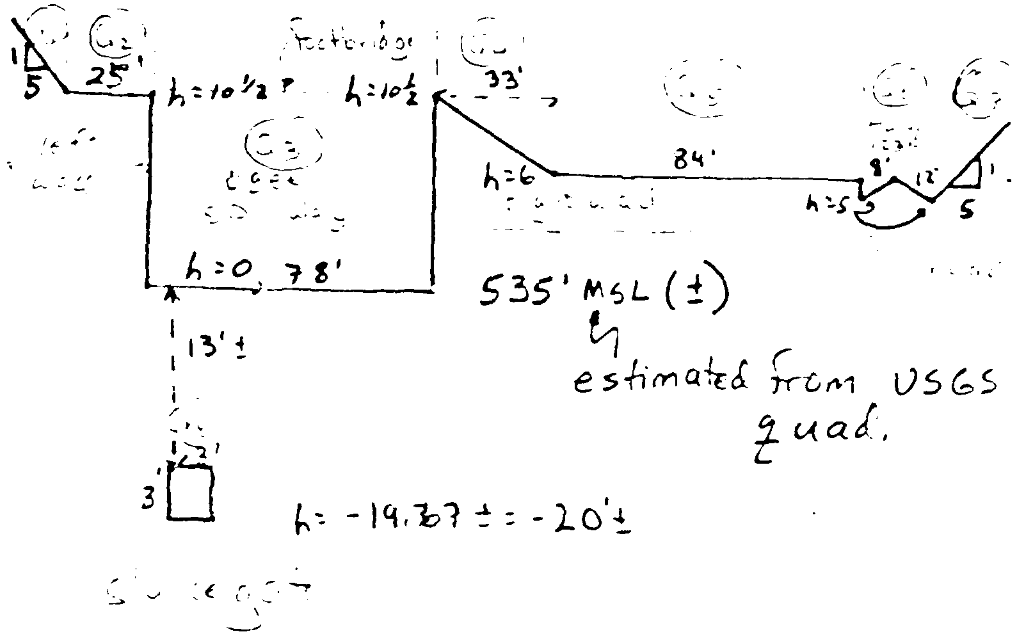
11. Channel approximately 300 yards downstream of dam

12. View of reservoir area from dam

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

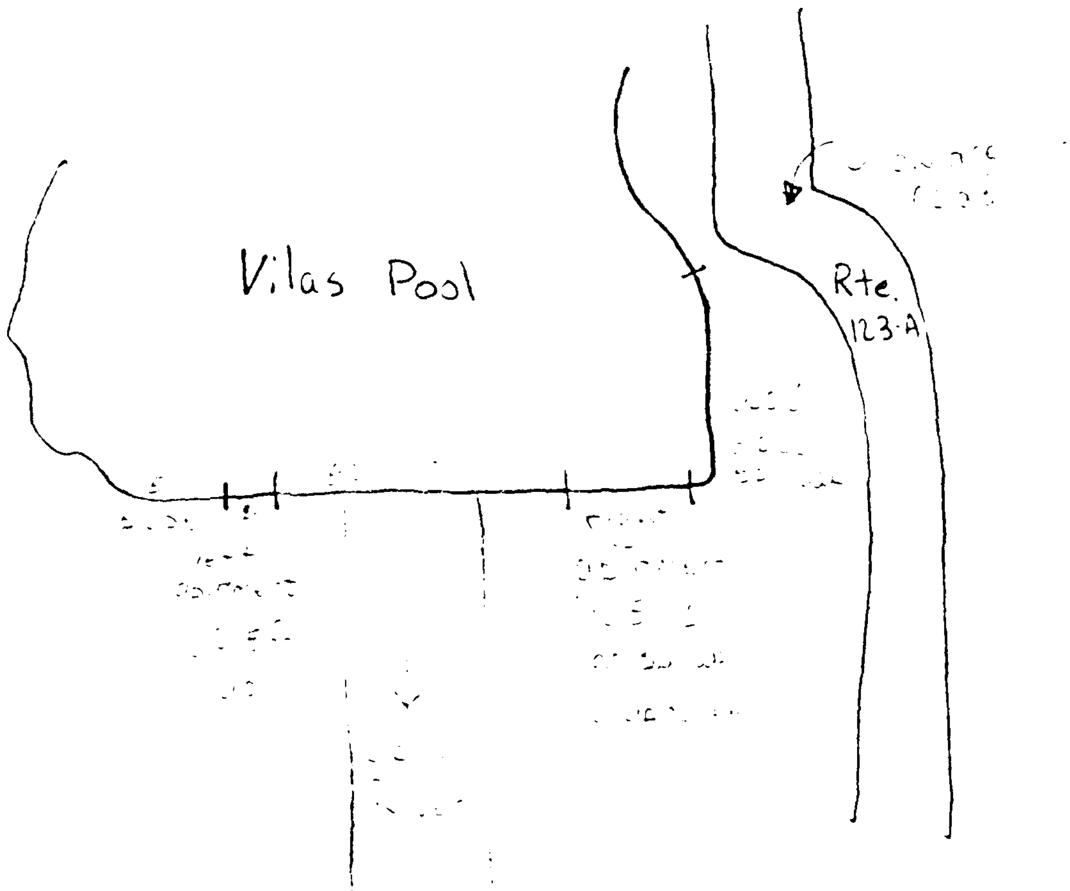
The information used to establish this elevation of Vilas Pool Dam was determined from field notes and an undated sketch of the dam:



The sluice gate is controlled by a mechanical valve operated from the footbridge over the spillway. At the time of inspection, this gate was partially open and difficult to operate. According to the representative of the city of Alstead, the gate is normally easier to operate. For the dam's "normal" stage/discharge curve, we will assume that this gate is ½ open. A curve for a fully open gate will also be developed.

There are two additional gates in the dam both inoperable. They are both 24" in diameter, & will be assumed to be closed.

Plan View of Vilas pool Dam.



h = elevation above spillway crest.

$h \leq 0$

Q_8 (1/2 open) | fully open

for an underflow sluice gate:

$$Q_8 = L\, b \sqrt{2 g h_s}\; C_d$$

C_d: Function of $\frac{b}{h}$

h_s: head above bottom of gate

$= h + 20$

b = height of gate = 1.5' for 1/2 open | $b = 3$

L = length of gate = 3

$$C_d = F\left[\frac{b}{h}\right] = F\left[\frac{1.5}{20}\right] \pm = F[0.08] = .59$$

$$C_d = F\left[\frac{3}{20}\right] = F[0.15] = .58$$

(p. 50, Rouse *Engineering Hydraulics*)

$$Q_8 = .59(3)(1.5)\sqrt{2g(h+20)} = 21.31(h+20)^{1/2}$$

$$Q_8 = .58(3)(3)\sqrt{2g(h+20)} = 41.89(h+20)^{1/2}$$

$Q_1 = Q_2 = Q_3 = Q_4 = Q_5 = Q_6 = Q_7 = 0$

$0 < h \leq 5$

$Q_3 = 3.7\,(78)(h)^{3/2}$

all others unchanged

$\boxed{C_d = 3.7 \text{ for ogee weir}}$

$5 \le h \le 6$

$Q_6 = 3.0(8(h-5))(.5(h-5))^{3/2} + 3(12(h-5))(.5(h-5))^{3/2}$

$Q_7 = 2.8(.5(h-5))(.5(h-5))^{3/2}$

All others unchanged

Cd = 3.0 broad concrete weir

Cd = 2.8 broad-crested earth weir

$6 < h \le 10.5$

$Q_6 = 3.0(20)(h-5.5)^{3/2}$

$Q_5 = 3.0(84)(h-6)^{3/2}$

$Q = 3.0(7.33(h-6))(.5(h-6))^{3/2}$

All others unchanged

$h > 10.5$

$Q_1 = 3.0(5(h-10.5))(.5(h-10.5))^{3/2}$

$Q_2 = 3.0(25)(h-10.5)^{3/2}$

$Q_4 = 3.0(33)(h-8.25)^{3/2}$

All others unchanged.

PP. 4-6 contain a BASIC program and the resulting calculations of the Stage - Discharge Curve for Vilas Pool Dam.

```
list
100 REM - STAGE/DISCHARGE CURVE FOR VILAS POOL DAM - GATE 1/2 OPEN
110 REM - STORED ON TAPE 5-1 FILE 6
120 PAGE
130 PRINT USING 140:
140 IMAGE10T"STAGE VS. DISCHARGE FOR VILAS POOL DAM - GATE 1/2 OPEN"
150 PRINT USING 160:
160 IMAGE //2T"HEAD"                    39T"DISCHARGE"
170 PRINT USING 180:
180 IMAGE 1T"(FEET)                      "32T"(CFS)"
190 PRINT USING 200:
200 IMA13T"TOTAL    SPILLWAY     GATE     ROAD     LEFT WALL     RIGHT WALL"
210 PRINT " "
220 FOR H=0 TO 20 STEP 0.5
230 Q1=0
240 Q2=0
250 Q3=0
260 Q4=0
270 Q5=0
280 Q6=0
290 Q7=0
300 Q8=21.31*(H-20)↑0.5
310 REM - sluice gate assumed 1/2 open
320 IF H<0 THEN 450
330 Q3=3.7*79*H↑1.5
340 IF H<5 THEN 450
350 Q6=3*(9*(H-5))*(0.5*(H-5))↑1.5+3*12*(H-5)*(0.5*(H-5))↑1.5
360 Q7=2.8*5*(H-5)*(0.5*(H-5))↑1.5
370 IF H<6 THEN 450
380 Q6=3*20*(H-5.5)↑1.5
390 Q5=3*84*(H-6)↑1.5
400 Q4=3*(7.33*(H-6))*(0.5*(H-6))↑1.5
410 IF H<10.5 THEN 450
420 Q1=3*5*(H-10.5)*(0.5*(H-10.5))↑1.5
430 Q2=3*25*(H-10.5)↑1.5
```

```
440 Q4=3*33*(H-8.25)↑1.5
450 T1=Q1+Q2+Q3+Q4+Q5+Q6+Q7+Q8
460 T2=Q1+Q2
470 T3=Q4+Q5
480 T4=Q6+Q7
490 PRINT USING 500:H,T1,Q3,Q8,T4,T2,T3
500 IMAGE      2D.2D,12D,10D,09D,08D,11D,14D
510 NEXT H
520 END
```

5.

STAGE VS. DISCHARGE FOR VILAS POOL DAM - GATE 1/2 OPEN

HEAD (FEET)	DISCHARGE (CFS) TOTAL	SPILLWAY	GATE	ROAD	LEFT WALL	RIGHT WALL
0.00	95	0	95	0	0	0
0.50	198	102	96	0	0	0
1.00	386	289	98	0	0	0
1.50	629	530	99	0	0	0
2.00	916	816	100	0	0	0
2.50	1242	1141	101	0	0	0
3.00	1602	1500	102	0	0	0
3.50	1993	1890	103	0	0	0
4.00	2413	2309	104	0	0	0
4.50	2860	2755	105	0	0	0
5.00	3333	3227	107	0	0	0
5.50	3835	3723	108	5	0	0
6.00	4376	4242	109	26	0	0
6.50	5055	4783	110	74	0	30
7.00	5854	5345	111	138	0	260
7.50	6742	5928	112	219	0	484
8.00	7714	6530	113	314	0	757
8.50	8764	7152	114	425	0	1073
9.00	9889	7792	115	551	0	1431
9.50	11067	8450	116	683	0	1828
10.00	12357	9136	117	849	0	2265
10.50	13693	9813	118	1022	0	2740
11.00	15154	10529	119	1210	37	3269
11.50	16700	11255	120	1415	80	3931
12.00	18328	11997	121	1636	152	4423
12.50	20035	12754	121	1974	242	5043
13.00	21819	13527	122	2128	349	5692
13.50	23678	14315	123	2400	472	6367

6

6A

14.00	25611	15113	134	2690	613	7067
14.50	27618	15975	125	[illegible]	779	7792
15.00	29698	16756	[illegible]	[illegible]	944	8540
15.50	31950	17611	[illegible]	[illegible]	1135	9311
16.00	34075	18470	[illegible]	[illegible]	1344	10105
16.50	36270	19347	[illegible]	[illegible]	1570	10920
17.00	38738	20229	[illegible]	[illegible]	1814	11756
17.50	41175	21123	[illegible]	[illegible]	2077	12613
18.00	43686	22043	[illegible]	[illegible]	2357	13489
18.50	46266	22964	[illegible]	[illegible]	2657	14366
19.00	48915	23902	[illegible]	[illegible]	2976	15301
19.50	51640	24851	124	[illegible]	[illegible]	16235
20.00	54437	25913	[illegible]	[illegible]	[illegible]	17188

STAGE VS. DISCHARGE FOR VILAS POOL DAM - GATE OPEN

HEAD (FEET)	DISCHARGE (CFS) TOTAL	SPILLWAY	GATE	ROAD	LEFT WALL	RIGHT WALL
0.00	187	0	187	0	0	0
0.50	292	102	190	0	0	0
1.00	481	289	192	0	0	0
1.50	724	530	194	0	0	0
2.00	1013	816	196	0	0	0
2.50	1339	1141	199	0	0	0
3.00	1701	1500	201	0	0	0
3.50	2093	1890	203	0	0	0
4.00	2514	2309	205	0	0	0
4.50	2962	2755	207	0	0	0
5.00	3436	3227	209	0	0	0
5.50	3939	3723	212	5	0	0
6.00	4491	4242	214	26	0	0
6.50	5162	4793	216	74	0	79
7.00	5951	5345	218	139	0	260
7.50	6950	5928	220	219	0	484
8.00	7823	6530	222	314	0	757
8.50	8874	7152	224	425	0	1073
9.00	10009	7792	226	551	0	1431
9.50	11189	8452	228	692	0	1828
10.00	12479	9126	229	849	0	2265
10.50	13812	9819	231	1022	0	2740
11.00	15369	10529	233	1210	27	3369
11.50	16816	11255	235	1415	80	3831
12.00	18445	11997	237	1636	152	4423
12.50	20153	12754	239	1874	242	5043
13.00	21937	13527	241	2129	349	5692
13.50	23797	14315	242	2400	472	6367

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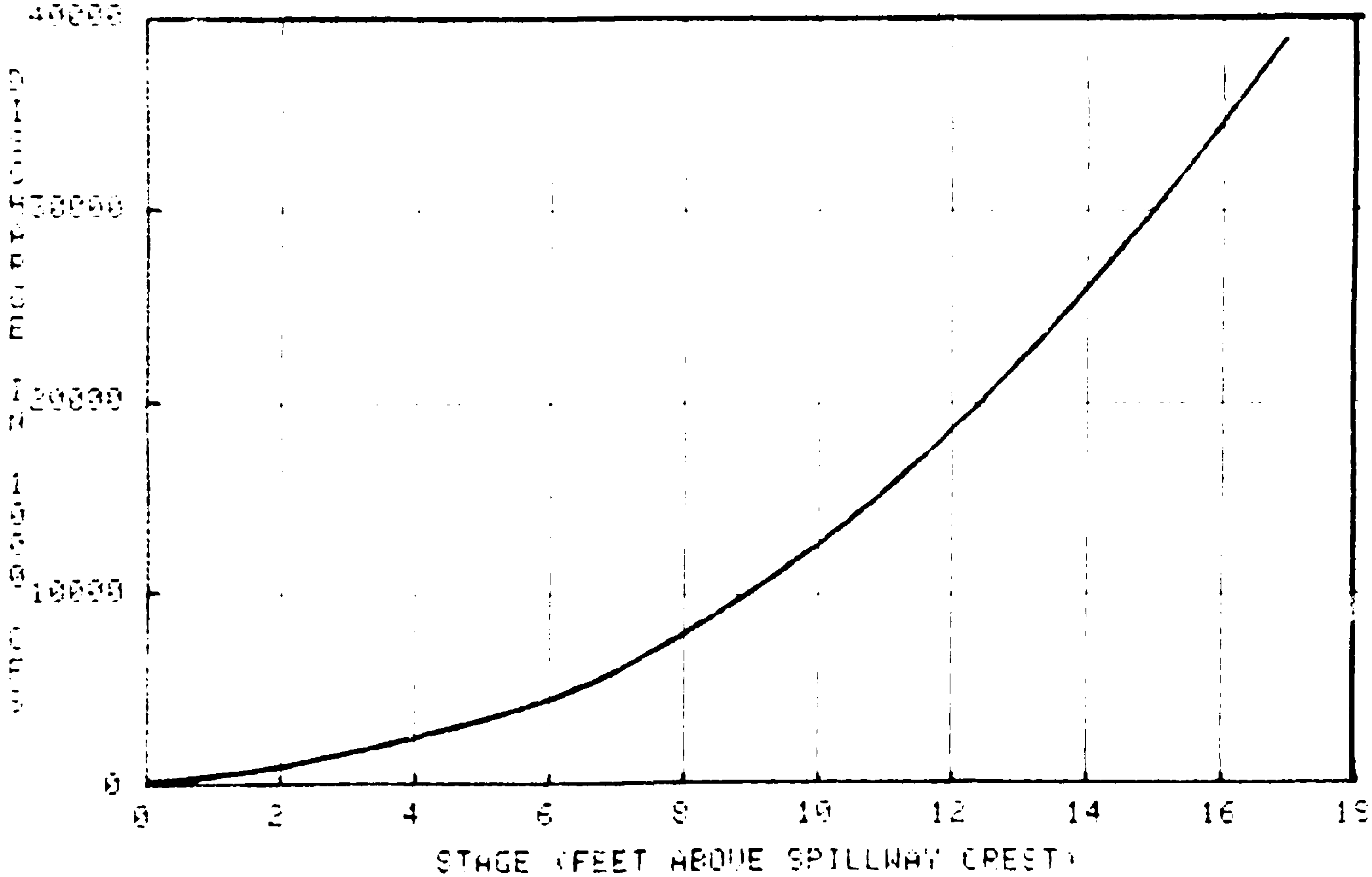
14.00	25731	15118	244	2690	613	7067
14.50	27779	15935	246	2997	770	7792
15.00	29820	16766	248	3322	944	8540
15.50	31973	17611	250	3665	1135	9311
16.00	34198	18470	251	4028	1344	10105
16.50	36495	19342	253	4403	1570	10920
17.00	38867	20229	255	4899	1814	11756
17.50	41302	21126	257	5229	2077	12613
18.00	43912	22040	258	5669	2357	13489
18.50	46394	22964	260	6127	2657	14386
19.00	49046	23902	262	6606	2976	15301
19.50	51769	24851	263	7106	3314	16235
20.00	54564	25813	265	7626	3671	17189

7a

STAGE/DISCHARGE CURVE FOR VILAS POOL DAM

P. 8



Stage - Storage Curve

Assuming a pond area of 6 acres and no spreading as the pond rises, surcharge storage is given by:

$$S = 6h$$

Total storage = $80 + 6h$

The Stage-Storage curve is given on p. 10.

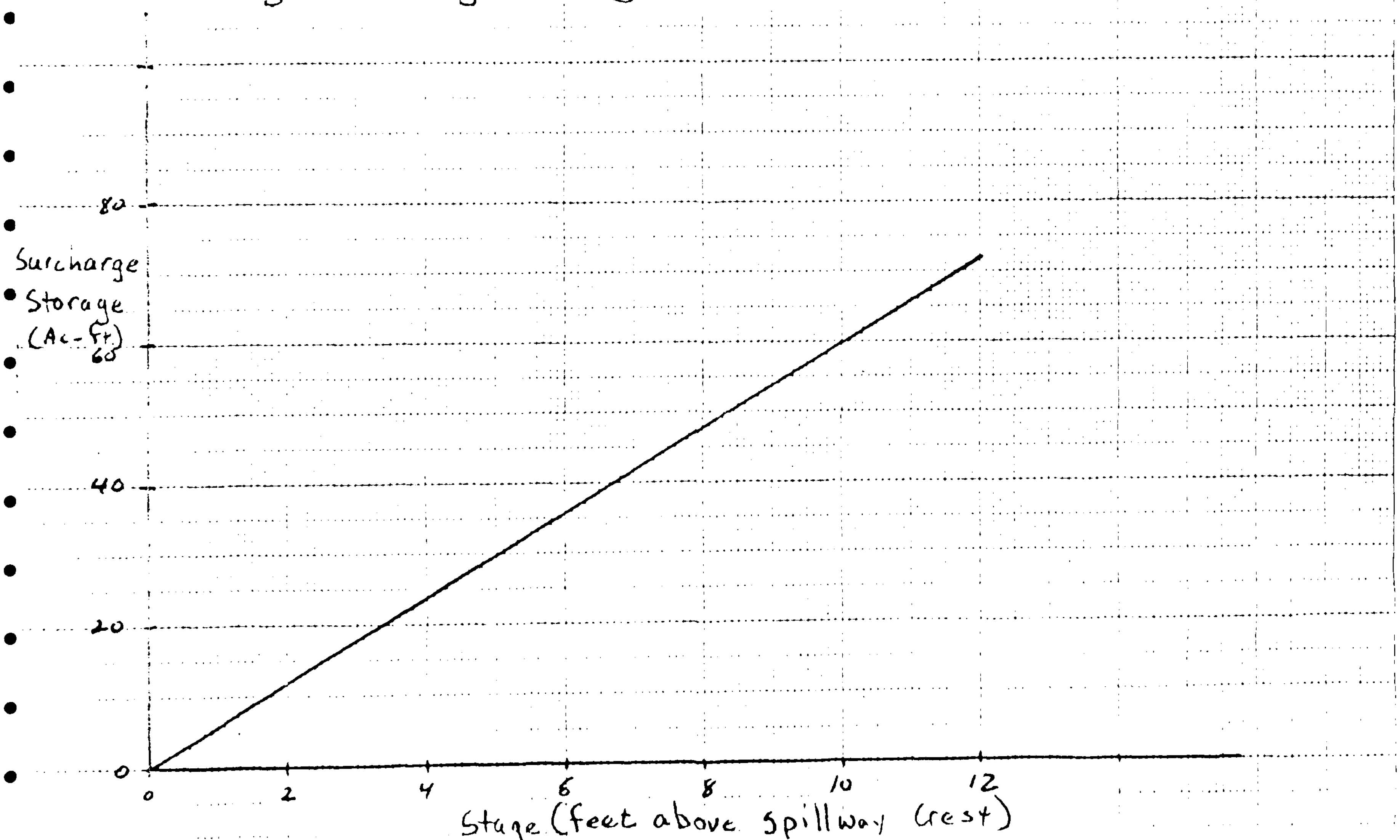
For the drainage area of 62 sq. mi.:

$$1'' \text{ of runoff} = \frac{1}{12}(62)(640) = 3307 \text{ ac-ft.}$$

$$1 \text{ Ac-ft} = \frac{1}{3307} = .0003024'' \text{ of runoff}$$

Surcharge storage to top of dam (6 ft. above s/w crest)
= 36 Ac-ft = .0109" of runoff.

Stage-Surcharge Storage Curve



Dam Failure Analysis

A location and downstream hazard area map for Vilas Pool Dam is given at the end of this appendix.

Assume that failure occurs when the water surface elevation reaches the top of the right wall of the dam, 6 ft. above the spillway crest. (541 ± ft. MSL). (Spillway crest is estimated as 535 MSL from USGS quad).

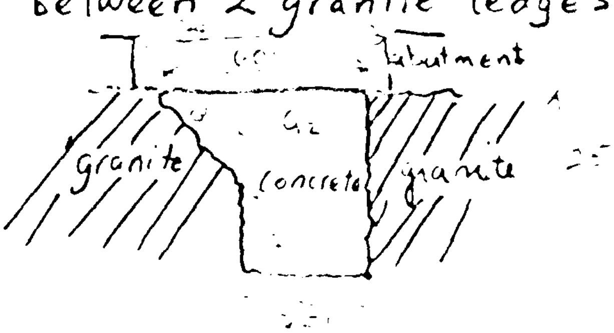
The pre-failure flow at this elevation is about 4400 cfs assuming the gate to be 1/2 open (4250 cfs over the spillway, 100 cfs through the gate, and 50 cfs the road).

Peak Dam Failure Flow = Normal Outflow + Breach Outflow

Normal outflow = 4400 cfs.

Breach Outflow = $Q_{p_1} = 8/27 \sqrt{g}\, W_b\, y_0^{3/2}$

where: W_b = breach width - normally 40% of width at 1/2 height of dam. In this case the spillway is set in between 2 granite ledges:



We will assume that failure results in failure of the entire spillway - leaving only the granite.

So: $Q_{p_1} = Q_1 + Q_2$

$$= 8/27\, w_{b_1}\, y_{o_1}^{3/2} \sqrt{g} + 8/27\, w_{b_2}\, y_{o_2}^{3/2} \sqrt{g}$$

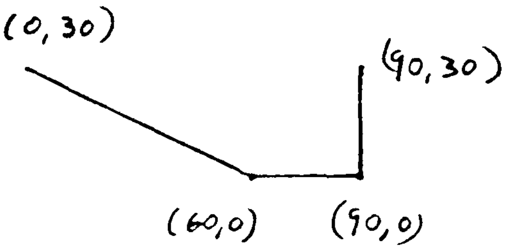
$w_{b_1} = 25'$

$w_{b_2} = 35'$

y_o: height above tailwater at failure.

Tailwater is controlled by the stream just downstream of the reservoir. The following typical stream section for the reach from Vilas Pool Dam for about 300 feet downstream is based on field notes and U.S.G.S. topo information.

$s = .011$
$n = .05$
$L = 300'$



A depth-normal flow relationship for this reach is given on p. 13. The pre-failure outflow of 9400 cfs would create 10.1 ft. of flow in this reach.

So, $y_{o_2} = 25 + 6 - 10.1 = 20.9$

$y_{o_1} = 6 + 6.5 = 12.5$ (6.5' is the average height from the spillway to the ledge.)

so $Q_{p_{1a}} = Q_1 + Q_2 = 8/27\,(25)\sqrt{g}\,(12.5)^{3/2} + 8/27\,(35)\sqrt{g}\,(20.9)^{3/2}$

p. 13

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	31.0	33.2	0.9	29.6	92.5
2.00	2.0	64.0	36.5	1.8	93.1	291.1
3.00	3.0	99.0	39.7	2.5	182.1	569.1
4.00	4.0	136.0	42.9	3.2	293.4	917.0
5.00	5.0	175.0	46.2	3.8	425.6	1330.0
6.00	6.0	216.0	49.4	4.4	577.7	1805.7
7.00	7.0	259.0	52.7	4.9	749.5	2342.6
8.00	8.0	304.0	55.9	5.4	940.8	2940.3
9.00	9.0	351.0	59.1	5.9	1151.5	3598.9
10.00	10.0	400.0	62.4	6.4	1381.7	4318.6
11.00	11.0	451.0	65.6	6.9	1631.7	5099.9
12.00	12.0	504.0	68.8	7.3	1901.7	5943.7
13.00	13.0	559.0	72.1	7.8	2191.9	6850.6
14.00	14.0	616.0	75.3	8.2	2502.6	7821.7
15.00	15.0	675.0	78.5	8.6	2834.1	8857.9
16.00	16.0	736.0	81.8	9.0	3186.8	9960.3
17.00	17.0	799.0	85.0	9.4	3561.1	11129.9
18.00	18.0	864.0	88.2	9.8	3957.1	12367.8
19.00	19.0	931.0	91.5	10.2	4375.4	13675.1
20.00	20.0	1000.0	94.7	10.6	4816.3	15053.0
21.00	21.0	1071.0	98.0	10.9	5280.0	16502.5
22.00	22.0	1144.0	101.2	11.3	5767.1	18024.9
23.00	23.0	1219.0	104.4	11.7	6277.9	19621.2
24.00	24.0	1296.0	107.7	12.0	6812.7	21292.7
25.00	25.0	1375.0	110.9	12.4	7371.9	23040.4
26.00	26.0	1456.0	114.1	12.8	7955.8	24865.5
27.00	27.0	1539.0	117.4	13.1	8564.9	26769.2
28.00	28.0	1624.0	120.6	13.5	9199.5	28752.6
29.00	29.0	1711.0	123.8	13.8	9860.0	30816.8
30.00	30.0	1800.0	127.1	14.2	10546.7	32963.1

REACH 1 - FIRST 300 FEET D/S OF DAM

1858 + 5620 = 7500 cfs

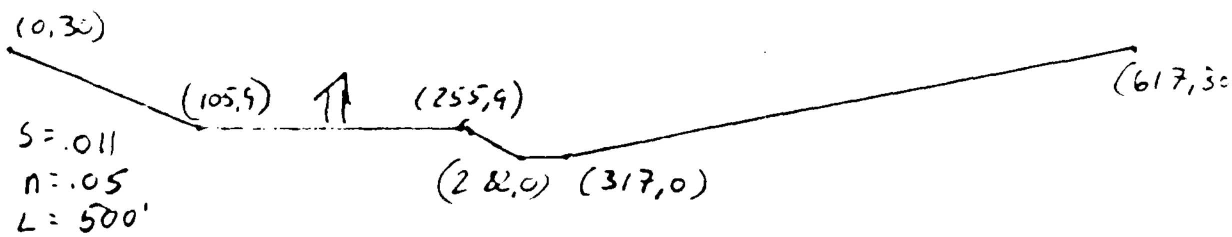
So peak dam failure outflow.

4400 + 7500 ≅ 11900 cfs

storage at failure = 80 + 6(6) = 116 Ac-ft.

This would increase the tailwater stage to 17.6 ft., which would not cause damage in the first 300 ft. reach. There would be little attenuation in this short, steep-walled reach.

About 300 ft. downstream of the dam, the Cold River passes into a wider floodplain for the 500 feet to the river's confluence with Warren Brook. In this reach there is one house on the east bank about 9 feet above the stream bed. Rte 123-A is also on the east bank about 9 feet above the streambed. The following typical cross-section for this reach is based on field notes and U.S.G.S. topo information:



The house is near the upstream end of this reach. According to the stage-normal flow relationship given on p. 15, the pre-failure flow of ~~4550~~ 4400 cfs would create

P.15

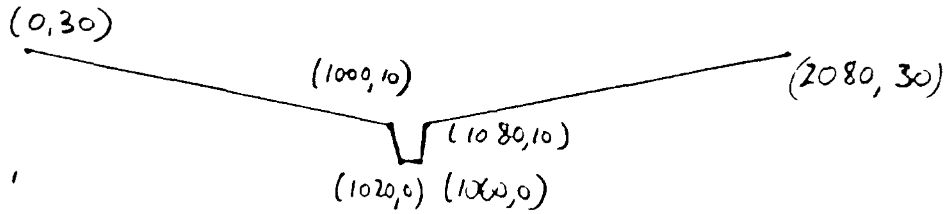
DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	41.5	48.2	0.9	37.6	117.4
2.00	2.0	96.0	61.4	1.6	129.3	404.1
3.00	3.0	163.5	74.6	2.2	275.9	862.2
4.00	4.0	244.0	87.8	2.8	482.3	1507.4
5.00	5.0	337.5	101.1	3.3	754.4	2357.7
6.00	6.0	444.0	114.3	3.9	1097.8	3431.2
7.00	7.0	563.5	127.5	4.4	1518.4	4745.8
8.00	8.0	696.0	140.7	4.9	2021.7	6318.8
9.00	9.0	841.5	153.9	5.5	2613.1	8167.2
10.00	10.0	1151.0	319.1	3.6	2708.5	8465.3
11.00	11.0	1475.5	334.2	4.4	3972.9	12417.0
12.00	12.0	1815.0	349.4	5.2	5447.4	17025.5
13.00	13.0	2169.5	364.5	6.0	7129.4	22282.7
14.00	14.0	2539.0	379.7	6.7	9018.2	28186.0
15.00	15.0	2923.5	394.8	7.4	11114.1	34736.6
16.00	16.0	3323.0	410.0	8.1	13418.3	41938.3
17.00	17.0	3737.5	425.1	8.8	15932.7	49795.8
18.00	18.0	4167.0	440.2	9.5	18659.4	58319.2
19.00	19.0	4611.5	455.4	10.1	21601.2	67513.5
20.00	20.0	5071.0	470.5	10.8	24760.8	77388.7
21.00	21.0	5545.5	485.7	11.4	28141.3	87954.2
22.00	22.0	6035.0	500.8	12.0	31745.8	99220.1
23.00	23.0	6539.5	516.0	12.7	35577.8	111196.6
24.00	24.0	7059.0	531.1	13.3	39640.5	123894.4
25.00	25.0	7593.5	546.3	13.9	43937.4	137324.2
26.00	26.0	8143.0	561.4	14.5	48472.1	151497.2
27.00	27.0	8707.5	576.6	15.1	53248.1	166424.4
28.00	28.0	9287.0	591.7	15.7	58269.0	182117.0
29.00	29.0	9881.5	606.9	16.3	63538.5	198586.3
30.00	30.0	10491.0	622.0	16.9	69069.0	215943.7

REACH 2 - 300 FEET DOWNSTREAM OF DAM TO CONFLUENCE WITH WARREN BROOK

a stage of 6.7 feet. ~~The~~ Dam failure would cause the stage to increase to 10.9 feet, causing 2 feet of flooding at the house and on Rte 123A. This would present a threat of loss of life. The attenuation of dam failure flows due to storage in this reach is calculated on p. 17.

The attenuated peak dam failure flow at the confluence with Warren Brook is 11,300 cfs, which yields a stage of 10.7 feet. Assuming an inflow of 700 cfs from Warren Brook, the prefailure flow in the Cold River downstream would be 5100 cfs, and the peak failure flow 12,000 cfs, a 6900 cfs increment due to failure.

For the next 3200 feet downstream of the confluence to the Rte. 123 Bridge, the Cold River runs through Alstead. The development in this reach is 15+ feet above the river. The following typical section for this reach is based on field notes and USGS topo information:



S = .016
n = .05
L = 3200'

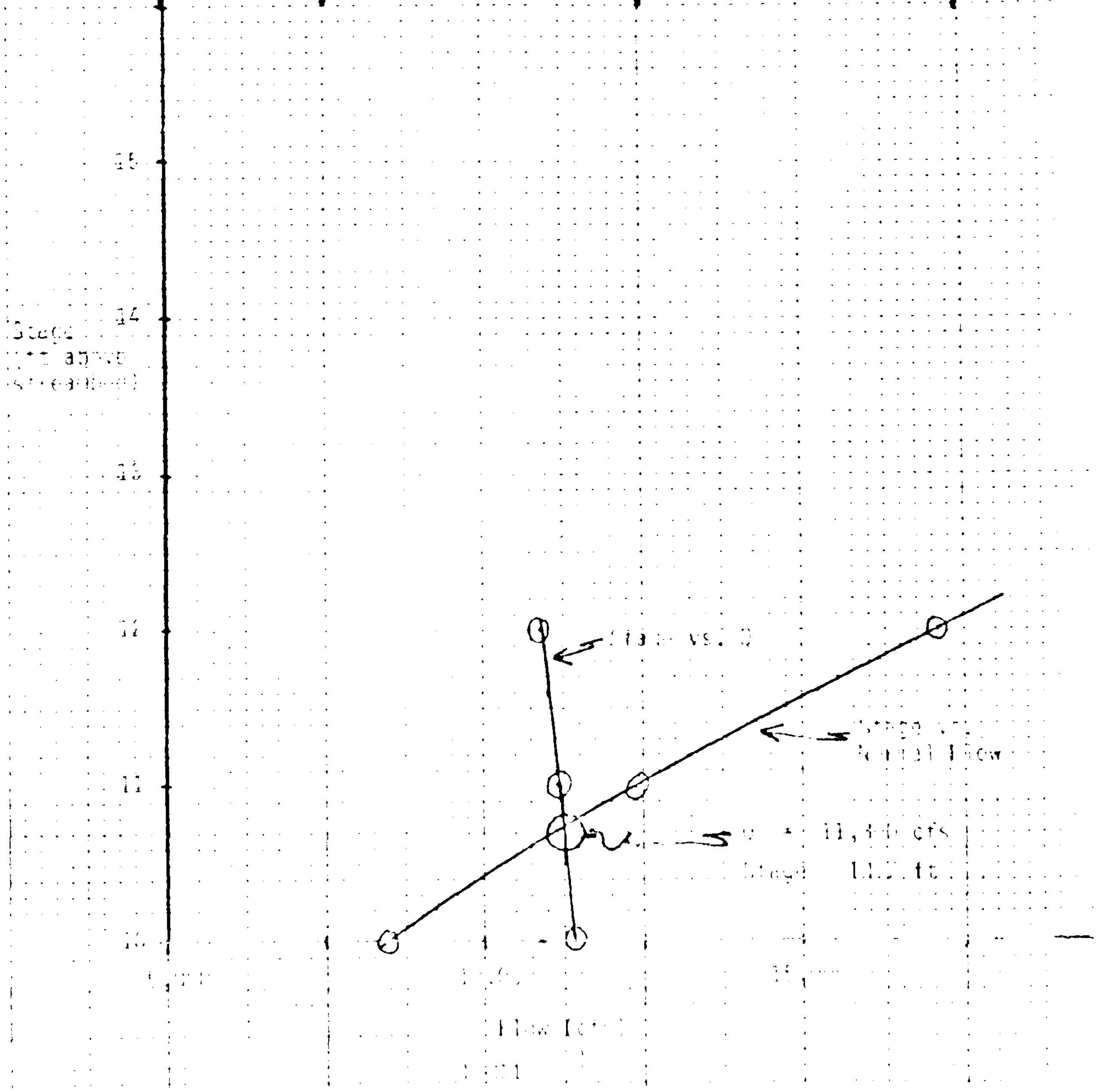
The attenuation due to storage in this reach is calculated on p. 19, based on the stage-normal flow relationship given on p. 18. The pre-failure flow of 5100 cfs would create a stage of 7.8 ft. in this reach.

Atten.ated Peak Dam Failure Flow at Confluence with Warren Brook   TCG, 9/5/79, p. [illegible]

Baseflow = 4400 cfs; Floodwave = 7500 cfs

$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{116}\right) = 7500\left(1 - \frac{STOR}{116}\right)$ $Q = 4400 + Q_{p1}$

Stage (ft)	Area (about [illegible]) (sq ft)	STOR $\left(\frac{AREA \times 500}{43{,}560}\right)$ (ac ft)	[illegible] (cfs)
10	600	7.2	11,8[illegible]
11	94[illegible]	10.9	11,[illegible]
12	12[illegible]	14.[illegible]	[illegible]



DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	42.0	44.5	0.9	40.4	152.4
2.00	2.0	88.0	48.9	1.8	130.1	490.6
3.00	3.0	139.0	53.4	2.6	259.9	979.7
4.00	4.0	192.0	57.9	3.3	427.2	1610.2
5.00	5.0	250.0	62.4	4.0	631.2	2379.2
6.00	6.0	312.0	66.8	4.7	871.9	3286.7
7.00	7.0	378.0	71.3	5.3	1149.9	4334.4
8.00	8.0	449.0	75.8	5.9	1465.6	5524.7
9.00	9.0	522.0	80.2	6.5	1820.1	6860.7
10.00	10.0	600.0	84.7	7.1	2214.1	8346.1
11.00	11.0	730.0	184.7	4.0	1825.4	6880.8
12.00	12.0	960.0	284.8	3.4	2159.3	8139.2
13.00	13.0	1290.0	384.8	3.4	2890.8	10896.6
14.00	14.0	1720.0	484.8	3.5	4002.7	15088.0
15.00	15.0	2250.0	584.8	3.8	5526.9	20833.4
16.00	16.0	2880.0	684.8	4.2	7507.0	28297.3
17.00	17.0	3610.0	784.9	4.6	9989.4	37654.2
18.00	18.0	4440.0	884.9	5.0	13020.1	49078.5
19.00	19.0	5370.0	984.9	5.5	16644.6	62740.9
20.00	20.0	6400.0	1084.9	5.9	20907.0	78807.5
21.00	21.0	7530.0	1184.9	6.4	25850.1	97440.1
22.00	22.0	8760.0	1285.0	6.8	31515.5	118795.8
23.00	23.0	10090.0	1385.0	7.3	37944.0	143027.5
24.00	24.0	11520.0	1485.0	7.8	45175.1	170284.7
25.00	25.0	13050.0	1585.0	8.2	53247.5	200713.0
26.00	26.0	14680.0	1685.0	8.7	62199.0	234454.9
27.00	27.0	16410.0	1785.1	9.2	72066.5	271649.9
28.00	28.0	18240.0	1885.1	9.7	82886.4	312434.9
29.00	29.0	20170.0	1985.1	10.2	94694.3	356943.9
30.00	30.0	22200.0	2085.1	10.6	107525.1	405308.9

P.18

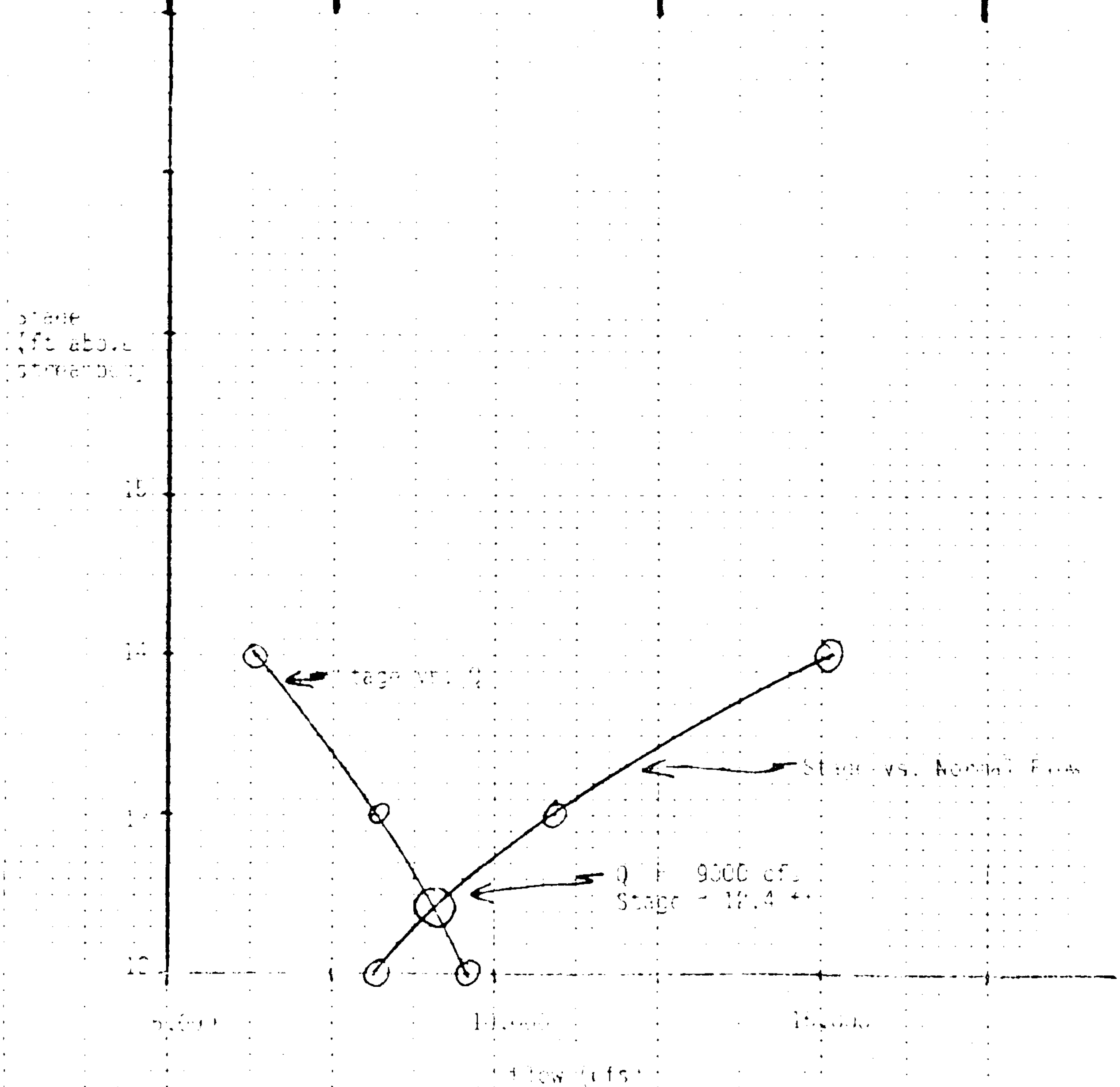
REACH 3 - CONFLUENCE WITH WARREN BROOK TO RTE. 123 BRIDGE

Attenuated Peak Dam Failure Flow at Rt. 12 Bridge TCG, 9/5/79, p. 14

Q = BASEFLOW + Q_{p2} BASEFLOW = 5100 cfs

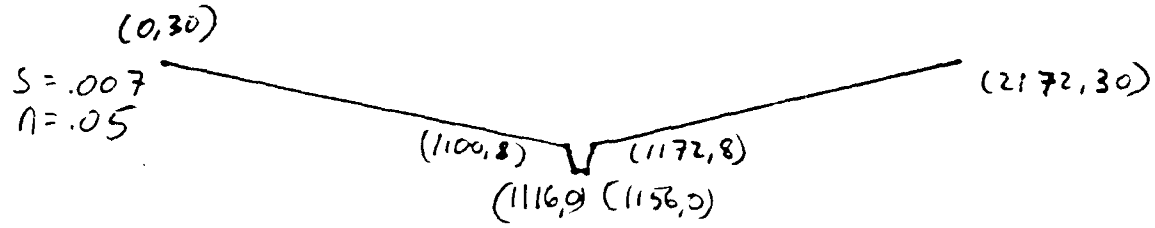
$Q_{p2} = Q_{p1}\,(1 - \frac{STOR}{110}) = 6900\,(1 - \frac{STOR}{110})$

Stage (ft)	Area (sq. ft.)	STOR = (AREA × 5280) / 43,560 (ac-ft)	Q (cfs)
10	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]
14	2300	95.8	10000



The attenuated peak dam failure outflow at the Rte. 123 bridge would be ~~5100~~ ~~8850~~ 9000 cfs, an increment of ~~3700~~ ~~3400~~ 3900 cfs due to dam failure. This would create a stage of ~~12.4~~ ~~12.3~~ 12.4 feet. The Rte. 123 bridge is a concrete structure with an 18' x 70' opening. It is unlikely to obstruct dam failure flows.

Downstream of the dam the Cold River flattens out considerably. In the first 1300 feet below the bridge there is also considerable development along the river, with about 10 houses with first floors 8 -12 feet above the channel. The following typical cross-section for this reach is based on field notes and USGS topographic information:



According to the Stage / Normal flow relationship given on p. 21, the pre-failure flow of ~~5500~~ 5100 cfs would create a stage of about ~~10.8~~ 10.7 feet in this reach, causing flooding of 0-3 feet. The peak flood flow of ~~9100~~ ~~8800~~ 9000 cfs would increase the stage by ~~2~~ 1½ feet to ~~12.5~~ 12.2 feet, increasing

P.21

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	42.0	44.5	0.9	40.4	100.8
2.00	2.0	88.0	48.9	1.8	130.1	324.5
3.00	3.0	138.0	53.4	2.6	259.9	648.0
4.00	4.0	192.0	57.9	3.3	427.2	1065.1
5.00	5.0	250.0	62.4	4.0	631.2	1573.7
6.00	6.0	312.0	66.8	4.7	871.9	2174.0
7.00	7.0	378.0	71.3	5.3	1149.9	2866.9
8.00	8.0	448.0	75.8	5.9	1465.6	3654.2
9.00	9.0	567.7	171.3	3.3	1262.7	3148.3
10.00	10.0	792.3	266.7	2.9	1605.6	4003.1
11.00	11.0	1093.5	362.2	3.0	2285.2	5697.5
12.00	12.0	1499.6	457.7	3.3	3309.6	8251.6
13.00	13.0	2001.2	553.2	3.6	4718.1	11763.4
14.00	14.0	2598.2	648.6	4.0	6556.2	16346.2
15.00	15.0	3290.6	744.1	4.4	8870.0	22115.2
16.00	16.0	4078.5	839.6	4.9	11704.8	29182.9
17.00	17.0	4961.9	935.1	5.3	15104.2	37658.5
18.00	18.0	5940.7	1030.5	5.8	19110.8	47647.9
19.00	19.0	7015.0	1126.0	6.2	23765.7	59253.7
20.00	20.0	8184.7	1221.5	6.7	29108.8	72575.4
21.00	21.0	9449.9	1317.0	7.2	35179.0	87709.9
22.00	22.0	10810.5	1412.4	7.7	42014.1	104751.5
23.00	23.0	12266.6	1507.9	8.1	49651.1	123792.5
24.00	24.0	13818.2	1603.4	8.6	58126.1	144922.8
25.00	25.0	15465.2	1698.9	9.1	67474.5	168230.5
26.00	26.0	17207.6	1794.3	9.6	77730.8	193802.2
27.00	27.0	19045.5	1889.8	10.1	88929.2	221722.4
28.00	28.0	20978.9	1985.3	10.6	101102.9	252074.6
29.00	29.0	23007.7	2080.8	11.1	114284.9	284940.5
30.00	30.0	25132.0	2176.2	11.5	128507.4	320400.7

REACH 4 - D/S OF RTE. 123 BRIDGE

the flooding correspondingly. Since the homes threatened by flooding might well be abandoned and since the increment generated by dam failure would be small, the threat of loss of life at this location is probably slight.

The attenuated peak dam failure flow and stage at the downstream end of this reach is calculated on p. 23. The peak dam failure flow would be 8500 ~~[illegible]~~ ~~[illegible]~~ cfs, which would create a stage of 12.10 feet (an increase of 1.34 feet over pre-failure conditions.) Below this reach, the river flows through several miles without development, in which dam failure flow would [illegible]

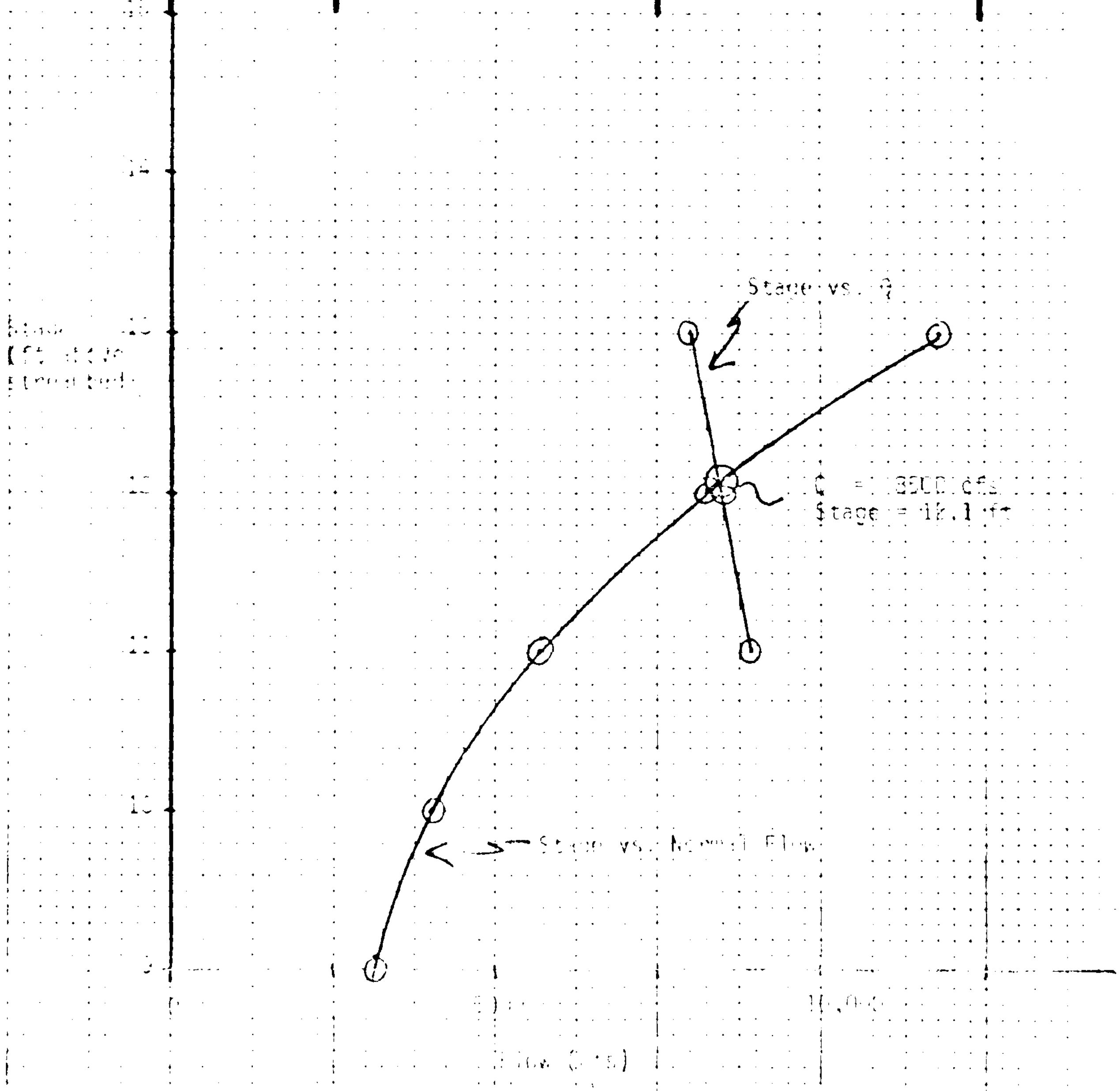
The chart on P. 24. summarizes the results of the failure of Vilas Pool Dam.

Attenuated Peak Dam Failure Flow 1300 Feet Downstream of Rt. 123 Bridge [illegible]

Q = Baseflow + Q_{p2} Baseflow = 5100 cfs

$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{[illegible]}\right) = 3900\left(1 - \frac{STOR}{11[illegible]}\right)$

Stage (ft)	Area (above 10.7 ft) (sq ft)	STOR $\left(\frac{AREA \times 13.1}{43,560}\right)$ (ac ft)	[illegible]
11	[illegible]	[illegible]	[illegible]
12	[illegible]	14.9	[illegible]
13	[illegible]	[illegible]	[illegible]



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Location	# of dwellings	level above Streambed (ft)	Flow and Stage Before Failure	Flow and Stage After Failure	Comments
tailwater	-	-	4400 cfs 10.1 ft.	11,900 cfs 17.6 ft.	
house, ~500 ft. d/s of dam	1	9	4400 cfs 6.7 ft	11,900 cfs 10.9 ft.	danger of loss of life. Floods 123-A.
just u/s of Warren Bk.	-	-	4400 cfs 6.7 ft.	11,300 cfs 10.7 ft.	
just d/s of Warren Bk.	-	-	5100 cfs 7.6 ft.	12000 cfs 13.2 ft.	
just u/s of Rte. 123 Bridge	-	-	5100 cfs 7.6 ft.	9000 cfs 12.4 ft.	
just d/s of Rte. 123 Bridge	10 ±	8-12	5100 cfs 10.7 ft.	9000 cfs 12.2 ft.	Slight danger of loss of life
1300' d/s of Rte. 123 Bridge			5100 cfs 10.7 ft.	8500 cfs 12.1 ft.	

Test Flood Analysis

Size Classification: Small (storage = S = 116 AF; height = h = 31'. 50 ≤ S ≤ 1000 AF; 25 ≤ h < 40')
Hazard Classification: High

The hazard classification is HIGH due to the potential for serious economic losses and some loss of life downstream in the event of dam failure (see chart, p. 24).

Test Flood: 1/2 PMF to PMF.

For situations in which a range of possible test floods is given, the COE "Suggested Guidelines" indicate that the value most closely relating to the dam's hazard classification should be used. Since Vilas Pool Dam is on the low side of high, we will use a test flood of 1/2 PMF.

Using the COE NED "Maximum Probable Flood Peak Flow Rates", the upstream drainage area of 62 square miles of rolling terrain would yield a PMF peak inflow of 1100 csm. 1/2 PMF = 550 csm

Peak Test Flood Inflow = 34,100 cfs.

For a drainage area of 62 sq. mi, the storage pool in Vilas Pool Reservoir has a negligible

attenuating effect. According to the stage-discharge curve, with the gate 1/2 open the peak flow of 34,100 cfs would create a stage of ~~15.4~~ 16 ft. above the spillway crest, ~~9.4~~ 10 ft. above the wall, & ~~5.4~~ ~~5.4~~ 5.5 feet above the top of the abutments.

The stage would remain essentially unchanged with the gate fully open or closed.

<u>Area relation for peak discharge.</u> - Area at Dam vs. area at USGS Drewsville gage.

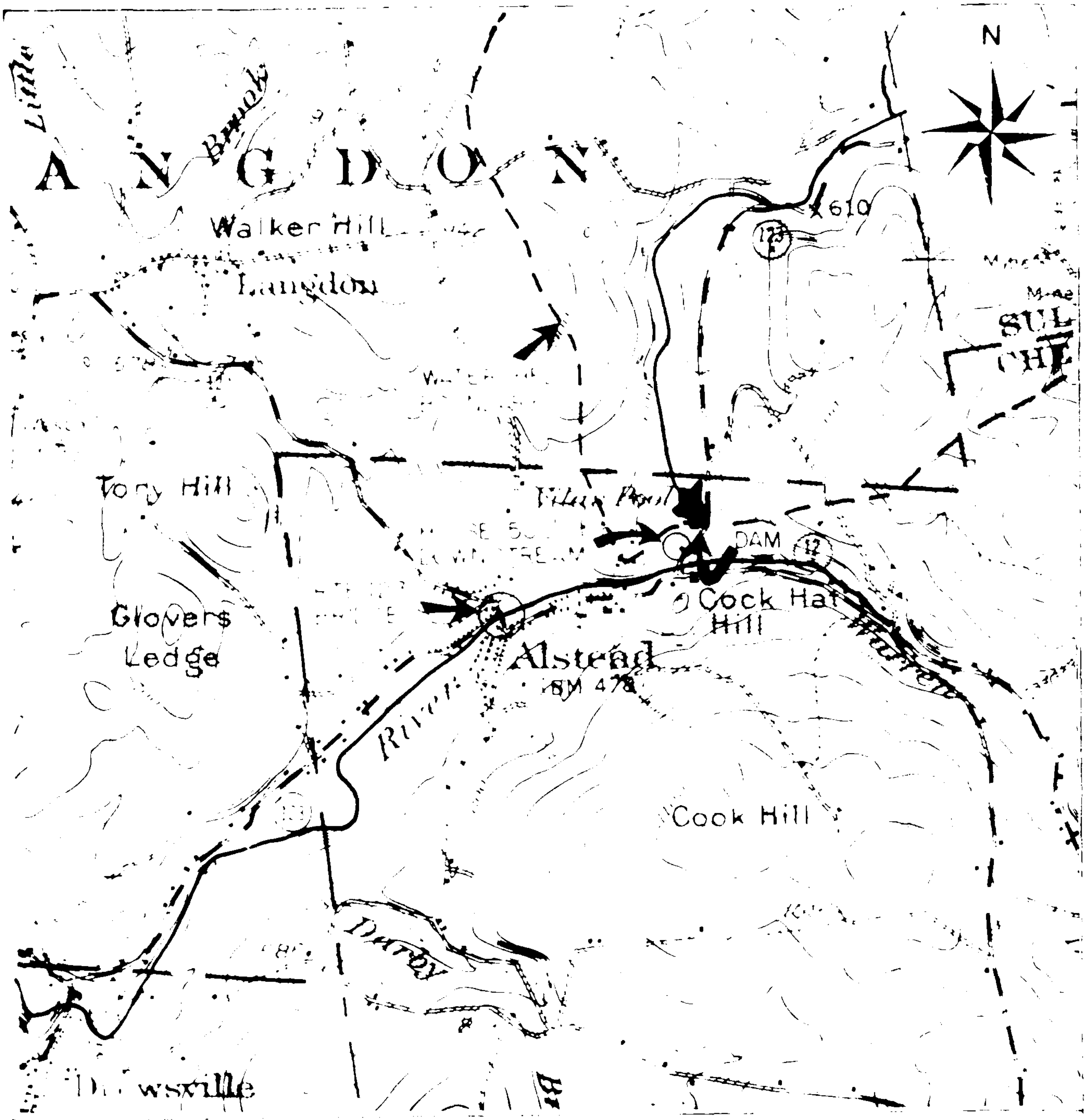
Peak at gage = 6710 cfs

Area at gage = 83 mi^2

Area at dam = 62 mi^2

$$\frac{Q_{P1}}{Q_{P2}} = \left(\frac{A_1}{A_2}\right)^{.75}$$

$$\text{peak at dam} = 6710 \left(\frac{.62}{.83}\right)^{.75} = 5390 \text{ cfs}$$



NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

LOCATION AND DOWNSTREAM HAZARD MAP

APPENDIX I

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST	STATE	COUNTY	CONGR DIST	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
NH	9	NED	NH	005	02				VILAS POOL DAM	4309.2	7220.9	09OCT79

POPULAR NAME	NAME OF IMPOUNDMENT
	VILAS POOL

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI)	POPULATION
01	08	COLD RIVER	ALSTEAD	1	600

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT)	HYDRAULIC HEIGHT (FT)	IMPOUNDING CAPACITIES MAXIMUM (ACRE FT)	NORMAL (ACRE FT)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
CTPGOT	1925	R	31	31	116	80	NED	N	N	N	N	

REMARKS

21-STONE MASONRY

D/S HAS	SPILLWAY CREST LENGTH	TYPE	WIDTH (FT)	MAXIMUM DISCHARGE (FT)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	PROPOSED (MW)	NO	NAVIGATION LOCKS LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)
1	136	U	78													

OWNER	ENGINEERING BY	CONSTRUCTION BY
TOWN OF ALSTEAD		

REGULATORY AGENCY

DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NHWRB	NHWRB	NHWRB	NHWRB

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
GOLDBERG ZOINO DUNNICLIFF + ASSOC	30AUG 9	PL92-367

REMARKS